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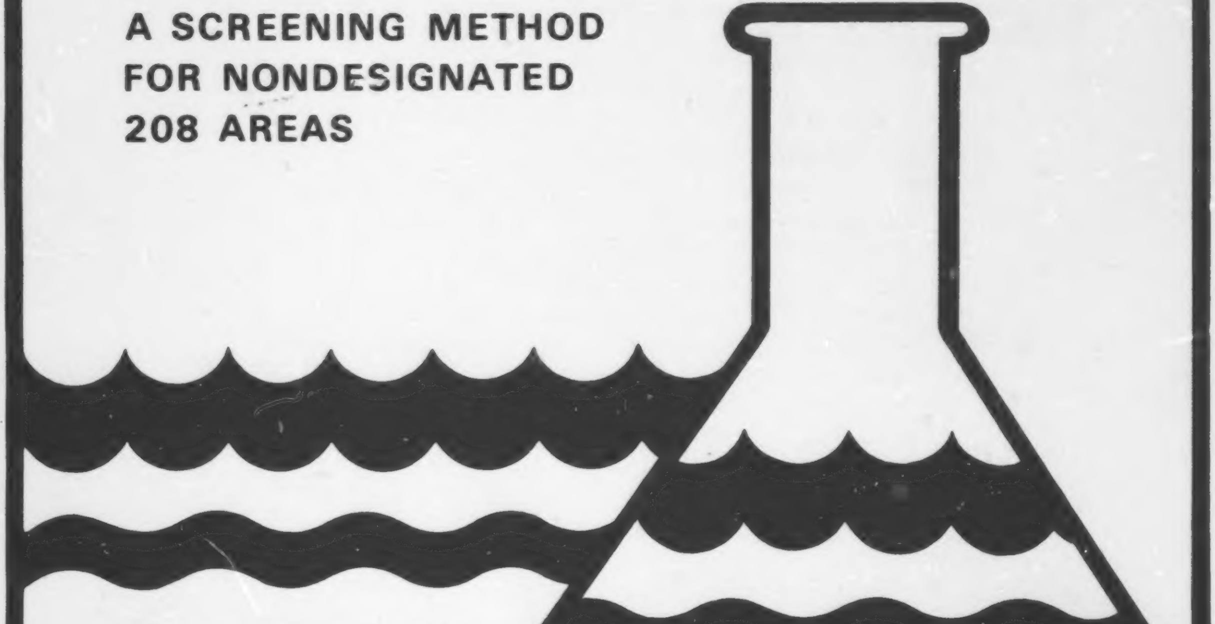

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A Screening Method for Nondesignated 208 Areas

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FOREWORD

As environmental controls become more costly to implement and the penalties of judgment errors become more severe, environmental quality management requires more efficient analytical tools based on greater knowledge of the environmental phenomena to be managed. As part of this Laboratory's research on the occurence, movement, transformation, impact, and control of environmental contaminates, the Technology Development and Applications Branch develops management or engineering tools to help pollution control officials achieve water quality goals through watershed management.

Basin planning requires a set of analysis procedures that can provide an assessment of the current state of the environment and a means of predicting the effectiveness of alternative pollution control strategies. This report contains a description of a set of consistent analysis procedures that accomplish these tasks. It is directed toward local or state government planners who must interpret technical information from many sources and recommend the most prudent course of action that will maximize the environmental benefits to the community and minimize the cost of implementation.

David W. Duttweiler
Director
Environmental Research Laboratory

ABSTRACT

The objective of this study is to develop a methodology for the preliminary screening of surface water quality applicable for use by nondesignated 208 planning agencies. Analytical methods are included for the assessment of rivers, impoundments, and estuaries. Additionally, methods are provided by which point and nonpoint sources can be evaluated.

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TABLE OF CONTENTS

Page

Foreword iii

Abstract iv

List of Figures viii

List of Tables xvii

Chapter

1 Introduction 1

- 1.1 Background 1
- 1.2 Problem Definition 2
- 1.3 Purpose and Scope 2

2 Overview of Methodology 4

- 2.1 Planning Steps 4
- 2.2 Base Map 5
- 2.3 Data Collection 5
- 2.4 Plotting Data on Maps 8
- 2.5 Analysis of Hydrologic Data, Estimation of Waste Loads, Data Supplementation 9
- 2.6 Stream Analyses 9
- 2.7 Impoundment Analyses 10
- 2.8 Estuarine Analyses 11
- 2.9 Presentation of Results 12
- 2.10 Limitations of Methodology 12

3 Waste Loading Calculations 14

- 3.1 Introduction 14
- 3.2 Sediment Loading Function for Surface Erosion 16
- 3.3 Nitrogen Loading Function 67
- 3.4 Phosphorus Loading Function 76
- 3.5 Organic Matter Loading Function 80
- 3.6 Accuracy of Nutrient and Organic Matter Loading Functions 84
- 3.7 Loading Values for Salinity Loads in Irrigation Return Flow 86
- 3.8 Pollutants from Urban Runoff 91

TABLE OF CONTENTS (Continued)

Chapter		Page
	3.9 Point Source Waste Loads	104
	References	123
4	Rivers and Streams	127
	4.1 Introduction	127
	4.2 Carbonaceous and Nitrogenous Oxygen Demand	141
	4.3 Dissolved Oxygen	160
	4.4 Temperature	196
	4.5 Nutrients and Eutrophication Potential	221
	4.6 Total Coliform Bacteria	234
	4.7 Conservative Constituents	240
	4.8 Sedimentation	244
	4.9 Presentation of Results	259
	References	266
5	Impoundments	270
	5.1 Introduction	270
	5.2 Impoundment Stratification	271
	5.3 Sediment Accumulation	289
	5.4 Eutrophication	332
	5.5 Impoundment Dissolved Oxygen	355
	5.6 Application of Methods and Presentation of Results: Some Recommendations	386
	References	393
6	Estuaries	395
	6.1 Introduction	395
	6.2 Present Water Quality Assessment	417
	6.3 Estuarine Classification	421
	6.4 Flushing Time Calculations	445
	6.5 Calculation of Pollutant Concentrations in Estuaries	463
	6.6 Thermal Pollution	506
	6.7 Turbidity and Sedimentation	518
	6.8 Tidal Prism/Throat Cross-Sectional Area Relationships	540
	References	544
Appendices		
A	Monthly Distribution of Rainfall Erosivity Factor R	A-1

TABLE OF CONTENTS (Continued)

Appendices		Page
B	Methods for Predicting Soil Erodibility Index k	B-1
C	River and Stream Data	C-1
D	Impoundment Hydrothermal Profiles for 10 Locations in U.S.	D-1
E	Hydrothermal Profile Simulation Method	E-1
F	Sediment Accumulation in Selected Impoundments in U.S.	F-1
G	Equivalents of Commonly Used Units of Measurement	G-1

LIST OF FIGURES

Figure		Page
III-1	Flow Diagram for Calculating Sediment Loading from Surface Erosion	19
III-2	Average Annual Values of the Rainfall--Erosivity Factor, R (U.S.D.A. and EPA, 1975)	23
III-3	Mean Annual Values of Erosion Index (in English Units) for Hawaii (U.S. Dept. of Agriculture, 1974)	24
III-4	Soil Moisture--Soil Temperature Regimes of the Western United States (U.S. Dept. of Agriculture, 1974)	25
III-5	Relationships between Annual Average Rainfall Erosivity Index and the 2-Year, 6-Hr. Rainfall Depth for Three Rainfall Types in the Western United States (U.S. Dept. of Agriculture, 1974)	28
III-6	Storm Distribution Regions in the Western United States (U.S. Dept. of Agriculture, 1974)	29
III-7	Slope Effect Chart Applicable to Areas A-1 in Washington, Oregon, and Idaho and All of A-3 (U.S. Dept. of Agriculture, 1974)	33
III-8	Slope Effect Chart for Areas Where Figure III-7 is not Applicable (U.S. Dept. of Agriculture)	34
III-9	Slope Effect Chart for Irregular Slopes (Forter and Wischmeier, 1973)	37
III-10	Sediment Delivery Ratio for Relatively Homogeneous Basins (McElroy, et al, 1976)	50
III-11	Projected Variation of Soil Erosion for Lands with Constant Cover Factor, in Parts of Michigan, Missouri, Illinois, Indiana, and Ohio (McElroy, et al, 1976)	62

LIST OF FIGURES (Continued)

Figure		Page
III-12	Projected Variation of Soil Erosion on Continuous Corn Lands in Central Indiana (McElroy, et al, 1976)	66
III-13	Percent Nitrogen (N) in Surface Foot of Soil (Parker, et al, 1946)	70
III-14	Soil Nitrogen vs. Humidity Factor and Temperature	72
III-15	Nomograph for Humidity Factor, H	73
III-16	Nitrogen (NH_4-N and NO_3-N) in Precipitation	75
III-17	Phosphorus Content in the Top 1 Foot of Soil (Parker, et al, 1946)	78
III-18	Climate Zone for the Cities from Which Data are Available and Used in the URS Study (Amy, et al, 1974)	97
III-19	Correlation between Population Density and Curb Length Density (American Public Works Association, 1975)	100
IV-1	Mechanisms of BOD Removal from Rivers	145
IV-2	Deoxygenation Coefficient as a Function of Depth, (After Hydroscience, 1971)	146
IV-3	Example of Computation of k_L from Stream Data (From Hydroscience, 1971)	148
IV-4	Hypothetical BOD Waste Loadings in a River	154
IV-5	Variability of Dissolved Oxygen by Season for 22 Major Waterways, 1968-72 (EPA, 1974)	161
IV-6	Reaeration Coefficient as a Function of Depth (From Hydroscience, 1971)	164
IV-7	Reaeration Coefficient for Shallow Streams, Owen's Formulation	166
IV-8	Flow Process of Solution to Dissolved Oxygen Problem in Rivers	173
IV-9	Nomogram for Solution to the Dissolved Oxygen Sag (After Thomas, 1948)	174

LIST OF FIGURES (Continued)

Figure		Page
IV-10	Characteristic Dissolved Oxygen Profile Downstream from a Point Source of Pollution	175
IV-11	Flow Process in Reach by Reach Solution to Critical Dissolved Oxygen Values	188
IV-12	Hypothetical River Used in Example IV-5	193
IV-13	Idealization of a Run-of-the-River Power Plant	198
IV-14	Mechanisms of Heat Transfer Across a Water Surface (Parker and Krenkel, 1969)	202
IV-15	Schematic of Site No. 3 Cooling Lake (From Edinger, et al, 1968)	203
IV-16	Observed Temperatures, Site No. 3, July 18-July 24, 1965 (Edinger, et al, 1968)	204
IV-17	Comparison of Computed Equilibrium and Ambient Temperatures with Observed Mean Diurnal Temperature Variations for Site No. 3 July 18-July 24, 1966 (Edinger, et al, 1968)	205
IV-18	Mean Daily Solar Radiation (Langleys) Throughout the United States for July and August (U.S. Department of Commerce, 1968)	207
IV-19	Downstream Temperature Profile for Completely Mixed Stream, $T-E/T_m-E$ vs. r (From Edinger, 1965)	219
IV-20	Flow Duration Curve, Hatchie River at Bolivar, Tenn. (From Cragwall, 1966)	222
IV-21	Frequency of Lowest Mean Discharges of Indicated Duration, Hatchie River at Bolivar, Tenn. (From Cragwall, 1966)	223
IV-22	Total Coliform Profiles for the Willamette River (EPA, 1974)	236
IV-23	Salinity Distribution in a Hypothetical River	244

LIST OF FIGURES (Continued)

Figure		Page
IV-24	Ψ and τ_c for DuBoys Relationship as Functions of Median Size of Bed Sediment (Task Committee on Preparation of Sedimentation Manual, 1971)	248
IV-25	Hydraulic Radii for Different Channel Shapes (From King, 1954)	250
IV-26	Sediment Discharge as a Function of Water Discharge for the Colorado River at Taylor's Ferry (Task Committee on Preparation of Sedimentation Manual, 1971)	257
IV-27	Sediment Discharge as a Function of Water Discharge for the Niobrara River at Cody, Nebraska (Task Committee on Preparation of Sedimentation Manual, 1971)	258
IV-28	Graphical Presentation of Results of Example IV-5	261
IV-29	Graphical Results of Example IV-4	263
V-1	Water Density as a Function of Temperature and Dissolved Solids Concentration (from Chen and Orlob, 1973)	272
V-2	Water Flowing into an Impoundment Tends to Migrate Toward a Region of Similar Density	273
V-3	Annual Cycle of Thermal Stratification and Overturn in an Impountment	275
V-4	Plots Used in Example V-1 Thermal Profile	287
V-5	Thermal Profile Plots Appropriate for Use in Example V-2	291
V-6	Sediment Rating Curve Showing Suspended Sediment Discharge as a Function of Flow (After Linsley, Kohler, and Paulhus, 1958)	294
V-7	A Relationship Between the Percent of Inflow-Transported Sediment Retained within an Impoundment and the Ratio of Capacity to Inflow (From Linsleyk Kohler, and Paulhus, 1958)	296

LIST OF FIGURES (Continued)

Figure		Page
V-8	Plot of C/R and CR^2 versus R (From Camp, 1968)	301
V-9	The Drag Coefficient (C) as a Function of Reynolds' Number (R) and Particle Shape (From Camp, 1968)	302
V-10	Schematic Representation of Hindered Settling of Particles in a Fluid Column	303
V-11	Velocity Correction Factor for Hindered Settling (From Camp, 1968)	305
V-12	Upper and Lower Lakes and Environs, Long Island, New York	310
V-13	Impoundment Configurations Affecting Sedimentation	314
V-14	Kellis Pond and Surrounding Region, Long Island, New York	317
V-15	Hypothetical Depth Profiles for Kellis Pond	318
V-16	Hypothetical Flow Pattern in Kellis Pond	319
V-17	Hypothetical Depth Profiles for Kellis Pond Not Showing Significant Shoaling	320
V-18	Lake Owyhee and Environs	322
V-19	New Millpond and Environs	323
V-20	Significance of Depth Measures D, D^1, and D^{11}, and the Assumed Sedimentation Pattern	326
V-21	Settling Velocity for Spherical Particles	327
V-22	Nomograph for Estimating Sediment Trap Efficiency	328
V-23	The Vollenweider Relationship	336
V-24	Plot of the Vollenweider Relationship Showing the Position of 133 Eastern U.S. Impoundments Impacted by Municipal Sewage Treatment Plant (MSTP) Effluents (National Eutrophication Survey, 1976)	337

LIST OF FIGURES (Continued)

Figure		Page
V-25	Relationship Between Summer Chlorophyll and Spring Phosphorus (from Lorenzen, unpublished)	343
V-26	Maximal Primary Productivity as a Function of Phosphate Concentration (After Chiandani, et al, 1974)	345
V-27	Conceptualization of Phosphorus Budget Modeling (Lorenzen, et al, 1976)	347
V-28	Geometric Representation of a Stratified Impoundment (From HEC, 1974)	358
V-29	Quality and Ecologic Relationships (From HEC, 1974)	359
V-30	Rate of BOD Exertion at Different Temperatures Showing the First and Second Deoxygenation Stages	364
V-31	Quiet Lake & Environs	373
V-32	Thermal Profiles for 20-Foot Deep Berlington, Vermont, Impoundment	381
V-33	A Possible Approach to Displaying Sediment Deposition Rates as a Function of Time and Axial Distance Downstream	389
V-34	Displaying Trap Efficiency as a Function of Grain Size	390
V-35	An Example of a Plot of DO Versus Time	391
VI-1	Typical Main Channel Salinity and Velocity Profiles for Stratified Estuaries	400
VI-2	Typical Main Channel Salinity and Velocity Profiles for Well Mixed Estuaries	402
VI-3	Typical Main Channel Salinity and Velocity Profiles for a Partially Mixed Estuary	403
VI-4	Estuarine Dimensional Definition	406

LIST OF FIGURES (Continued)

Figure		Page
VI-5	Saturation Temperature Salinity Relationships (Green, 1963)	408
VI-6	Schematic Diagram of Estuarine Water Quality Calculation Process	415
VI-7	Estuarine Classification Schematic	423
VI-8	Examples of Estuarine Suitability for Hand Calculations	424
VI-9	Estuarine Circulation-Stratification Diagram	426
VI-10	Examples of Estuarine Classification Plots (From Hansen and Rattray, 1966)	426
VI-11	Circulation and Stratification Parameter Term Diagram	428
VI-12	The Typical Estuary	429
VI-13	Typical Estuary Data for Classification Calculations	431
VI-14	Estuarine Circulation-Stratification Diagram	434
VI-15	Alsea Estuary Seasonal Salinity Variations (From Giger, 1972)	436
VI-16	Estuary Cross-Section for Tidal Prism Calculations	439
VI-17	Alsea Estuary Summer High Tide Salinity Profile	451
VI-18	Alsea Estuary Fraction of Fresh Water Segmentation	451
VI-19	Alsea Estuary Cross-Sectional Areas	452
VI-20	Alsea Estuary Cross-Sectional Areas	458
VI-21	Alsea Estuary Upstream Intertidal Volume	458
VI-22	Dispersion Methodology Schematic	465
VI-23	River Borne Pollutant Concentration for One Tidal Cycle	471

LIST OF FIGURES (Continued)

Figure		Page
VI-24	Alsea Estuary River Borne Conservative Pollutant Concentration	473
VI-25	Pollutant Concentration from an Estuarine Outfall	476
VI-26	Alsea Estuary Salinity and f_x Profile	477
VI-27	Alsea Estuary Example Heavy Metal Concentration for Two Outfalls	480
VI-28	Example Estuarine Non-Conservative Pollutant Concentration	488
VI-29	Incremental BOD Loading--DO Response in Estuaries	488
VI-30	Contributing Factors to Dispersion Coefficients in the Estuarine Environment	492
VI-31	Additive Effect of Multiple Waste Load Additions	495
VI-32	Single Event Pollutant Flow into an Estuary	500
VI-33	Estuarine Heat Budget (After Parker and Krenkel, 1970)	508
VI-34	Initial Thermal Plume Mixing	512
VI-35	Mean Suspended Solids in San Francisco Bay (From Pearson, et al, 1967, pg V-15)	522
VI-36	Sediment Movement in San Francisco Bay System (Million Cubic Yards) (From U.S. Engineering District, San Francisco, 1975)	529
VI-37	Idealized Estuarine Sedimentation	530
VI-38	Particle Diameter vs Settling Fall per Tidal Cycle (12.4 hrs) Under Quiescent Conditions (Spheres with density 2.0 gm/cm^3)	535
VI-39	Estuarine Null Zone Identification	538
VI-40	Tidal Prism vs Cross-Sectional Area, Regression Curves for all Inlets	542

LIST OF FIGURES (Continued)

Figure		Page
VI-41	Tidal Prism vs Cross-Sectional Area, Regression Curves for Inlets with One or No Jetties	543
VI-42	Tidal Prism vs Cross-Sectional Area, Regression Curves for Inlets with Two Jetties	544

LIST OF TABLES

Table		Page
III-1	Applicability of R_r and R_s Factors in the Areas West of the Rocky Mountains (U.S. Dept. of Agriculture, 1974a)	26
III-2	Relative Protection of Ground Cover Against Erosion (In Order of Increasing C Values)	41
III-3	"C" Values for Permanent Pasture, Rangeland, and Idle Land (Wischmeier, 1972)	42
III-4	"C" Values for Woodland (Wischmeier, 1972)	44
III-5	"C" Values for Construction Sites (Water Resources Administration, 1973)	45
III-6	"P" Values for Erosion Control Practices on Croplands (U.S. Dept. of Agriculture, 1974a)	46
III-7	Typical Values of Drainage Density	52
III-8	Summary of Applicability of Characteristic Factors	53
III-9	Estimated Range of Accuracy of Sediment Loads from Surface Erosion	56
III-10	Nutrient and Sediment Losses (Viets, 1971)	69
III-11	Sediment Yield in Example	82
III-12	Available Nitrogen Loading (Lb/Day)	82
III-13	Available Phosphorus Loading (Lb/Day)	83
III-14	Organic Matter Loadings (Lb/Day)	84
III-15	Probable Range of Loading Values for Nutrients and Organic Matter	85
III-16	Salt Yields from Irrigation in Green River Subbasin (EPA, 1971)	87

LIST OF TABLES (Continued)

Table		Page
III-17	Salt Yields from Irrigation in Upper Colorado Main Stream Subbasin (EPA, 1971)	88
III-18	Salt Yields from Irrigation in San Juan River Subbasin (EPA, 1971)	89
III-19	Salt Yields from Irrigation in Lower Colorado River Basin (EPA, 1971)	90
III-20	Salt Yields from Irrigation for Selected Areas in California (Water Resources Council, 1971)	90
III-21	Solid Loading Rates and Composition--Nationwide Means and Substitutions of the Nationwide Means at 80% Confidence Level (Amy, et al, etc.)	92
III-22	Mean Concentrations of Mercury and Chlorinated Hydrocarbons in Street Dirt from Nine U.S. Cities (Amy, et al, 1974)	94
III-23	Equivalent Curb-Length per Unit Area of Street Surface, Arranged by Land Use Types (Amy, et al, 1974)	101
III-24	General Land Consumption Rates for Various Land Uses (American Public Works Association, 1974)	101
III-25	Typical Municipal Waste Concentrations (Metcalf and Eddy, 1972)	106
III-26a	Examples of Municipal Discharges	107
III-26b	Point Source Loadings of Six Major Wastewater Treatment Facilities in One North Carolina 208 Area	108
III-27	Municipal Wastewater Flow Rates	109
III-28	Municipal Effluent Concentrations Corresponding to EPA Best Available Technology (BAT) Treatment Levels (Tetra Tech, Inc., 1975)	109
III-29	Characteristic Values for Urban Stormwater and Sewer Overflow Water Quality (Field, 1975)	111
III-30	Comparison of Quality of Storm Sewer Discharges for Various Cities (Lager and Smith, 1974)	112

LIST OF TABLES (Continued)

Table		Page
III-31	Comparison of Quality of Combined Sewage for Various Cities	113
III-32	Summary of Stormwater Pollutant Concentrations (Kaiser Engineers, 1969)	114
III-33	Typical Industrial Discharge Pollutant Concentrations	115
III-34a	Selected San Francisco Area Industrial Dischargers	116
III-34b	Selected San Francisco Area Industrial Dischargers	117
III-35	Summary of Current and Projected Waste Loads in One Region 208 Area (By SIC)	118
III-36	Typical Industrial Effluent Concentrations BPT - "1977" Approximate Mean Effluent Characteristics (EPA, 1974)	119
III-37	Typical Industrial Effluent Concentrations BAT - "1983" Approximate Mean Effluent Characteristics (EPA, 1974)	120
III-38	EPA BAT Guidelines for Selected New Source Industrial Discharges (30 Day Average)	121
IV-1	Reference Level Values of Water Quality Indicators for U.S. Waterways (EPA, 1974)	129
IV-2	Condition of Eight Major Waterways (EPA, 1974)	130
IV-3	Water Quality Problem Areas Reported by States Number Reporting Problems/Total (EPA, 1975)	133
IV-4	Water Quality Parameters Commonly Monitored by States (EPA, 1975)	134
IV-5	Annual Phosphorus and Nitrogen Load for Selected Iowa River Basins (EPA, 1975)	137
IV-6	Major Waterways: Seasonal and Flow Analysis, 1968-72 (EPA, 1974)	140
IV-7	Interrelationships between Sections of Chapter 4	142

LIST OF TABLES (Continued)

Table		Page
IV-8	Municipal Waste Characteristics before Treatment (Thomann, 1972)	143
IV-9	Solubility of Oxygen in Water (Standard Methods, 1971)	169
IV-10	D_c/L_o Values Versus D_o/L_o and k_a/k_L	178
IV-11	$k_a t$ Versus D_o/L_o and k_a/k_L	179
IV-12	Some Average Values of Gross Photosynthetic Production of Dissolved Oxygen (After Thomann, 1972, and Thomas and O'Connell, 1966)	183
IV-13	Chlorophyll a and Assimilation Number of Various Communities (After Odum, et al, 1958)	184
IV-14	Average Values of Oxygen Uptake Rates of River Bottoms (After Thomann, 1972)	185
IV-15	Compilation of Information in Example IV-4	190
IV-16	Critical Travel Time Results, Example IV-4	192
IV-17	Net Long Wave Atmospheric Radiation, H_{an}	209
IV-18	Water Vapor Pressure (mmHg) Versus Air Temperature, T_a, and Relative Humidity	210
IV-19	B and C(B) as Functions of Temperature	211
IV-20	Summary of Solar-Radiation Data for Mineola, Brookhaven, and the Connetquot River Sites	213
IV-21	Eutrophication Potential as a Function of Nutrient Concentrations	226
IV-22	Predicted Mean Total Phosphorus (TP) Concentrations (mg/l) (From EPA, 1976)	229
IV-23	Predicted Total Nitrogen (TN) Concentrations (mg/l) (From EPA, 1976)	230
IV-24	Total Nitrogen Distribution in a River in Response to Point and Non-Point Source Loading	233

LIST OF TABLES (Continued)

Table		Page
IV-25	Total Coliform Analysis (EPA, 1976)	235
IV-26	Salinity Distribution in a Hypothetical River	243
IV-27	Relationship of Total Suspended Sediment Concentration to Problem Potential (After EPA, 1976)	246
IV-28	Sediment Grade Scale (Task Committee on Preparation of Sedimentation Manual, 1971)	249
IV-29	Computing D/T for Determining the Hydraulic Radius of a Parabolic Section (From King, 1954)	251
IV-30	Relationship between Width to Depth Ratio of a Graded Stream and the Suspended and Bed Load Discharge (After Fenwick, 1969)	252
IV-31	Characteristics of the Colorado and Niobrara Rivers (Task Committee on Preparation of Sedimentation Manual, 1971)	255
IV-32	Summary of Preliminary Water Quality Analysis of the Hypothetical Rappahan River from Rivermile 280 to Rivermile 40	265
V-1	Parameter Values Used in Generation of Thermal Gradient Plots (Appendix C)	279
V-2	Temperature, Cloud Cover, and Dew Point Data for the Ten Geographic Locales Used to Develop Thermal Stratification Plots (Appendix D)	280
V-3	Limpid Lake Characteristics	286
V-4	Physical Characteristics of Lake Smith	288
V-5	Comparison of Monthly Climatologic Data for Shreveport, Louisiana,and Atlanta, Georgia	290
V-6	Hypothetical Physical Characteristics of Upper Lake, Brookhaven, Suffolk County, New York	311

LIST OF TABLES (Continued)

Table		Page
V-7	Hypothetical Physical Characteristics of Lower Lake, Brookhaven, Suffolk County, New York	313
V-8	Hypothetical Physical Characteristics (Assuming an Epilimnion Depth of 10 Feet) of Lower Lake, Brookhaven, Suffolk County, New York	315
V-9	Sources and Sinks of Impoundment Dissolved Oxygen	356
V-10	Oxygen Demand of Bottom Deposits (After Camp, 1968)	366
V-11	Saturation DO Levels in Fresh Water as a Function of Temperature (Chruchill, et al, 1961)	368
V-12	Characteristics of Quite Lake	374
V-13	Water Quality and Flow Data for Tributaries to Quiet Lake	374
V-14	Precipitation and Runoff Data for Quiet Lake Watershed	377
V-15	DO Sag Data for Quite Lake Hypolimnion	386
VI-1	Tidal Prisms for Some U.S. Estuaries	440
VI-2	Alsea Estuary Salinity and Volume by Segments	452
VI-3	Alsea Estuary Segmentation for Modified Tidal Prism Method	460
VI-4	Calculation of Segment Conservative Pollutant Quantity for Alsea Estuary	470
VI-5	Conservative Pollutant Concentrations above an Estuarine Outfall	479
VI-6	Comparison of Pollutant Concentrations for Alternate Outfalls in Alsea Estuary	479
VI-7	Typical Values for Decay Reaction Rates 'k'	482

LIST OF TABLES (Continued)

Table		Page
VI-7a	Temperature Correction for Selected Temperatures	483
VI-8	Tabulation of Values for Non-Conservative Pollutant Concentration	486
VI-9	kE Values for Several Types of Estuaries	493
VI-10	E Values for Selected Estuary (From Hydroscience, 1971)	493
VI-11	Gradiation of Candle Turbidimeter	521
VI-12	Maximum Allowable Channel Velocity to Avoid Bed Scour (FPS) (King, 1954)	527
VI-13	Sediment Particle Size Ranges (After Hough, 1957)	533
VI-14	Rate of Fall in Water of Spheres of Varying Radii and Constant Density of 2^{a} as Calculated by Stokes' Law^{b} (Mysels, 1959)	534
VI-15	Throat Area--Tidal Prism Relationships	542

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

One of the primary and potentially far reaching concepts set forth in the Federal Water Pollution Control Act Amendments of 1972 (PL 92-500) is contained in Section 208 of that Act. As originally proposed, this section was to facilitate the development and implementation of waste treatment management plans at the areawide level. Congress envisioned 208 as a process of creating comprehensive regional water quality planning to replace state planning in areas of severe water quality problems. The areal scope of 208 planning was originally limited to areas characterized by high urban-industrial concentrations, generally standard metropolitan statistical areas (SMSA's). Areas contiguous to SMSA's having substantial water quality problems could also be included.

On July 25, 1975 the U.S. District Court for the District of Columbia handed down a final decision which increased state responsibility for 208 planning to include nondesignated areas as well. This meant that 208 planning would be accomplished in all areas of each state, and not just those defined as "designated". Additionally this ruling obligated EPA to fund state programs for planning in the nondesignated areas. The Court decision set November 1, 1978 as the final deadline for submission to EPA of complete 208 plans for nondesignated areas. Although the 208 program was largely stalled during the first two and one-half years after the passage of the 1972 Act, it is now a high priority program and should continue as such through Phase II (1977-83) of planning.

1.2 PROBLEM DEFINITION

The areal extent of nondesignated 208 areas is extensive. Consequently, each area may contain a large number, as well as a wide spectrum, of surface water bodies. Additionally, data available to assess the severity of existing problems are often severely limited. Hence, there exists a need to facilitate the assessment of water quality problems occurring within these areas. By performing a preliminary analysis, areas of potential problems can be identified and screened from those areas where problems do not exist. Using more sophisticated techniques (e.g., computer models) the planner can then accurately and efficiently assess suspected problem areas. A subsequent report by Tetra Tech will provide this more advanced methodology.

1.3 PURPOSE AND SCOPE

This report contains a simplified methodology which nondesignated 208 planners can use to perform a preliminary assessment of surface water quality. Analyses require little data, and in most cases, can be accomplished with the assistance of a desk top calculator. Desk top calculation procedures are provided for the following subject categories:

1. Wasteload estimation, including point and non-point source pollutants.
2. Stream analyses for water temperature, biochemical oxygen demand, dissolved oxygen, total suspended solids, coliform bacteria, plant nutrients, and conservative constituents.
3. Lake analyses for thermal stratification, sediment accumulation, phosphorus budget, eutrophication potential, and hypolimnion DO.

4. Estuarine analyses for estuarine classification, temperature, BOD, DO, turbidity, sediment accumulation, and conservative constituents (e.g., salinity).

This report is intended to be used with little external input. Consequently, abundant data are included as tables, figures, and appendices. This is meant to increase the day-to-day utility of the document.

Where instructive, introductory material has preceded the actual presentation of water quality assessment methodologies. This was done to orient the planner toward pertinent background material, as well as to clearly state limitations of the methodologies due to assumptions and simplifications. Further, example calculations of the major emphases within each chapter are included to illustrate the ideas being presented. These examples are designed to unify the theory that has preceded it, as well as in some cases to introduce new, but related ideas.

The units most commonly used in this report are those that historically appear in the literature. Most often, the units are not metric. Consequently an english-metric conversion appendix is included at the end of this report.

CHAPTER 2

OVERVIEW OF METHODOLOGY

2.1 PLANNING STEPS

The initial 208 planning process should proceed with the following steps:

1. Preparation of a base map, delineating the planning area and the water system composed of lakes, streams and estuaries.
2. Data collection
3. Identification and mapping of land uses, population centers and industrial complexes.
4. Analysis of hydrologic data for annual runoff, monthly runoff, and critical flow for different parts of the water system.
5. Estimation of waste loads including point and non-point sources of pollutants.
6. Supplementation of data base.
7. Analysis of stream water quality and comparison of the results to water quality criteria and/or acceptable water quality objectives.
8. Analysis of lake water quality and evaluation of their acceptability.
9. Analysis of estuarine water quality.

This sequence of analysis is designed to follow the natural progression of a water system from upstream to downstream. Thus, estimates of waste loads are made to provide necessary input to streams, lakes, and estuaries. Some waste loads may enter a stream which feeds a lake, or an estuary. In this case, the stream analysis provides additional inputs to the calculations for the downstream water body.

2.2 BASE MAP

The first step in the planning process can be to obtain large scale topographic maps of the study area. These can be used to determine which water bodies are to be examined and to establish an order of study. Once this has been done, selected small scale (7 1/2 minute or 15 minute series) topographic sheets can be obtained. On these the planner can locate and mark point source discharges, regions of specific kinds of land use, population centers, and industrial complexes. Use of overlays or push pins may be helpful in preparing these displays.

The maps are also very important in showing the relationships among water bodies and the flow patterns for stormwater runoff. Finally, control strategies may be displayed for examination on the maps.

2.3 DATA COLLECTION

Once the base maps are prepared, the kinds of data needed should be fairly clear in most cases. In general terms, data needs will fall into two categories. These are hydrologic data and waste loading data.

2.3.1 Hydrologic Data

Hydrologic data includes such items as:

- Runoff quantity and quality
- Stream flows (low flows, statistical flows such as $7Q_{10}$, critical flows to be protected as decreed by law)

- Impoundment residence times, inflows and outflows, stagnant regions, stratification, internal flow patterns

- Estuarial tidal prism, flushing time

Much of the necessary hydrologic data will be available from the USGS, state geological survey, state environmental protection agencies, and other governmental organizations. In addition, data may be available from the private sector, as from universities, local citizens groups, and private firms.

Hydrologic data must usually be analyzed to serve as a basis for subsequent water quality analyses. Statistical methods may be applied to determine the annual runoff, monthly runoff and critical flow for a stipulated return frequency, on a selected time basis.

To select critical flow, for example, one must have some base knowledge of the seasonal distribution of stream flow and quantity - quality relationships. In general, the summer low flow is considered as the critical condition for stream and estuarine analyses. Average annual runoff is to be used for lake analyses, even though wet years are generally more critical from the standpoint of lake water quality.

To counter the lack of gaged hydrologic data, runoff may be estimated from rainfall information either through the rational method using a hand calculation or through a computerized runoff model such as EPA's SWMM or ARM.

Another type of data which may be needed is climatology data. Generally these are available from the National Climatic Center in Ashville, North Carolina. This agency can provide data summaries of various kinds for a large number of weather stations. Data include precipitation, cloud cover, humidity and other important parameters. Also, computer tapes can often be provided.

2.3.2 Waste Loads

Waste loads can usually be categorized as follows:

1. Point sources from cities and industries that discharge waste water at a number of concentrated points to the waterway.

2. Non-Point sources with diffuse areas of entry. These include:

   - Urban stormwater runoff
   - Agriculture drainage
   - Animal farms and feedlot drainage
   - Forest and other natural watershed runoff.

For municipal point sources, the flow and mass emission rate of various pollutants can best be obtained by requesting information directly from responsible city officials.

For industrial discharges, it is often difficult to obtain data. Many plant managers are reluctant to release data for a number of reasons including protection of proprietary processes. Accordingly, the planner often must seek sources of information other than the industrial plant itself. Sources include NPDES permits and process description references. The latter can be used in conjunction with work force figures to estimate waste loads. A similar approach may be used to estimate municipal discharge levels where data are unavailable. Here population data are a good indicator of raw sewage production, and the type of treatment (e.g., secondary treatment, trickling filter, holding pond) provides an estimate of the treatment efficiency. Also, national average wastewater characteristics are available for different categories of population centers and industries. Care must be exercised, however, to determine the potential errors of applying national averages to local situations.

For non-point sources, information is generally not available. Estimates may be made, however, using loading functions. These are discussed later in this report.

After wasteload estimates are made, it is important to display the magnitudes and entry points of the various waste loads on the base map. The display will provide a perspective for the problems at hand.

2.3.3 Other Data Categories

Many other kinds of data may be of use to the planner. These include, for example, in situ stream water quality data, rates of sediment accumulation in impoundments, and heavy metal concentrations in estuaries. Note that these are the areas to which the hand calculations developed in later chapters are addressed. It cannot be overemphasized that where data are already available, these should be used. Methods to be presented below are only meant to supplement existing data so that the planner can analyze his study region. Accordingly, all of the types of data implicit in methods presented below are candidates for the planner's data collection efforts.

2.4 PLOTTING DATA ON MAPS

Following data collection, the planner should plot important data on the base maps. Colored push pins (map pins) are especially useful here. As an example, where in situ data have been obtained, black tacks can represent areas of known high BOD levels. Coliform problems can be denoted by yellow pins, and DO problems by blue pins. Color coding may be functional. Green pins can represent algal blooms while white (sterility) can denote regions of very high heavy metals concentrations.

Point sources, such as industrial discharges can be denoted by numbered pins keyed to an accompanying detailed discharge quantity and quality index.

Other data not amenable to affixing to maps should not be neglected. These should be summarized carefully in tabular form. Such data might include various rates and coefficients, detailed population data, thermal profiles and other kinds of data which either cannot be associated with a specific geographic location or are too extensive to place on a map.

2.5 ANALYSIS OF HYDROLOGIC DATA, ESTIMATION OF WASTE LOADS, DATA SUPPLEMENTATION

In preparation for analysis, the planner should examine the available data to decide on the methods to be used. Generally, many analytical needs will be revealed during the data collection process itself. For example much DO data are likely to be available where severe DO problems are believed to exist. Other needs will be revealed upon inspection of the data. For example, where a particular point source is high in BOD, BOD and DO analysis are desirable. This, in turn, points to specific methods presented in later chapters. Use of those, in turn, should lead the planner either to the conclusion that no problem exists, or that further, more sophisticated analysis is necessary.

2.6 STREAM ANALYSES

While there are numerous other water quality parameters that may be considered important, analyses for the following parameters can usually pinpoint important problems rapidly.

- Biochemical Oxygen Demand
- Dissolved Oxygen
- Temperature
- Nutrients
- Total Coliforms

- Conservative Constituents, (total dissolved solids), and
- Suspended sediment and bed load

Basically, the stream analyses are performed in the following manner.

1. Selection of a critical flow for planning.
2. Segmentation of the river into reaches for calculation.
3. Determination of in-stream water quality concentrations based on the mass balance consideration, taking into account dilution, point sources, non-point sources, and decay.
4. Comparison of the resulting water quality concentrations against the water quality concentration objectives.

The water quality problem areas identified should be displayed on the base map. Such a display can provide a quick overview of areas requiring more detailed analysis.

2.7 IMPOUNDMENT ANALYSES

Delineation of water quality problems in lakes and impoundments, requires as a minimum, analysis of the following:

- Thermal stratification
- Sediment Accumulation
- Nutrient Budget - Eutrophication
- Hypolimnion Dissolved Oxygen

A computer model was used to calculate the thermal regimes of some hypothetical lake configurations in various geographic locations and under their respective typical climatological conditions. The planner can select the appropriate temperature profile by simply matching applicable conditions.

Sedimentation rates for a large number of impoundments have been studied by the U.S. Soil Conservation Service. Again, the planner can obtain answers by use of a look-up table, or he can make estimates of sedimentation rates based on several computational approaches.

The nutrient budget of an impoundment is basically a mass balance problem. Based on nutrient balance and hydraulic characteristics, the eutrophic state of an impoundment may be ascertained by the use of empirical curves. Such curves are presented in Chapter 5.

Because of the accumulation of oxygen-consuming materials and the lack of reoxygenation mechanisms in the hypolimnion, undesirable anoxic conditions may develop. Evaluation of hypolimnion dissolved oxygen requires an estimate of:

1. Initial DO at the onset of thermal stratification
2. Initial BOD at the onset of thermal stratification
3. Benthic BOD
4. DO sag over time

The final section of Chapter 5 provides a means to evaluate hypolimnion DO.

2.8 ESTUARINE ANALYSES

The complexity of estuarine hydrodynamics requires the use of computer models for a definitive water quality analysis. Only a limited number of problems are amenable to hand calculations. Such calculations, in many instances, help focus on problem areas for subsequent computer analysis.

Simple calculations that can be performed for an estuarine system include:

1. A gross estimate of mean water quality concentration.

2. Estuary classification. This can be accomplished based on some simple rules so as to define the applicability of subsequent hand calculation methods.

3. Flushing time calculations to determine the length of time required to replace the existing water mass within an estuary, or within a segment of an estuary. The total concentration of pollutants in the estuary may then be estimated.

4. Dispersion calculations for conservative and non-conservative pollutant concentrations. These groups include heavy metals, hydrocarbons, oil and greases, BOD, DO, nutrients, and coliform bacteria, etc.

5. Temperature calculation for heated discharges.

6. Analysis of sediment budget, sediment transport, its effect on turbidity, and location of critical sedimentation areas.

2.9 PRESENTATION OF RESULTS

It is important to emphasize the need for graphical presentation of analytical results. A simple display of when and where water quality problems are expected to occur for BOD, DO, temperature, heavy metals and other parameters can help decision-makers bring the problems and their potential solutions into focus. Such a display can be made on a base map in concert with accepted planning methods

2.10 LIMITATIONS OF METHODOLOGY

Factors that can influence system water quality include waste loads, climatology, hydrology, hydrodynamics, biology, chemistry and others.

Very complex phenomena occur in any real water system. Many assumptions and simplifications are clearly necessary for hand calculation methods.

Users of this manual must be aware of the limitations and the potential errors associated with the methodologies proposed and utilized. This tool is to be used only as a screening procedure and not as a method for definitive water quality assessment.

CHAPTER 3

WASTE LOADING CALCULATIONS

3.1 INTRODUCTION

This chapter outlines basic procedures that can be used to generate estimates of diffuse (non-point) and point-source loading rates. Loading functions for the following pollutants will be considered for non-point sources:

- Sediment
- Nutrients (phosphorus and nitrogen)
- Organic matter
- Salinity in irrigation return flow
- Urban related pollutants

Both cultivated and noncultivated agricultural land as well as urban areas are addressed in this report. For agricultural lands, sediment loading is calculated by use of the Universal Soil Loss Equation (USLE) (Section 3.2). This method has been adopted for a number of reasons, principal among them being the large data base that exists for the terms in the USLE. The loading of both nutrients and organic matter can be quantitatively related to sediment loading and this is done in this report. Thus, the discussion of nutrients and organic matter (Sections 3.3 through 3.5) logically follows the sediment loading calculations. These selected water quality parameters were chosen for inclusion in this report because they represent commonly occurring problems of major concern to planners.

Salinity in irrigation return flow is important in many areas in the arid western states. Considerable data are included in this report especially for the Colorado River basin, and the irrigated regions in California (Section 3.7).

For urban areas (Section 3.8) solids loading rates are first calculated and then the loading rates of other pollutants are related to them. Pollutants considered in this section include BOD, phosphorus, nitrogen, coliforms, and heavy metals.

Each of the major divisions on nonpoint source calculations (agriculture, salinity, and urban areas) in this chapter is essentially independent of every other. Accordingly, they can be used in any order. Within each section, the calculations performed can be used in two different ways. First, the magnitude of loadings can be compared for various alternatives (e.g., different land use schemes) to ascertain the significance of the changes. Second, the loadings can be used in calculations presented in Chapter 4 to assess the water quality impacts of nonpoint source pollutants on river and streams.

Finally, Section 3.9 presents estimates of typical point source pollutant loads for municipal and industrial discharges. Whenever possible, however, local data should always be used, if available, in lieu of the "typical" loadings given here.

The developments for non-point source loading presented in this chapter have been largely taken from a recent handbook developed by the Midwest Research Institute (McElroy, et al., 1976). That document provides supplemental background information on each of the loading functions presented here. For a more comprehensive presentation of the material in this chapter it is suggested the MRI handbook be consulted.

The units in which the loading functions are expressed need minor modifications for use in the non-point source loading expressions used in Chapter 4. Generally the loading functions in this chapter are expressed in units of kg/year or lb/year. In Chapter 4 the distributed loading terms are in units of lb/unit length/unit time, where the unit time may be, for example, a day and the length (in miles) is measured along the longitudinal axis of a water course. In general a change in the units in which time is expressed, as well as a division of the loading function by the number of river miles over which the loading occurs, will produce the proper units for use in Chapter 4.

3.2 SEDIMENT LOADING FUNCTION FOR SURFACE EROSION

Sediment loading is defined in this report as the quantity of soil material that is eroded and transported into the watercourse. Sediment loading is dependent on (a) on-site erosion, and (b) delivery, or the ability of runoff to carry the eroded material into the receiving waters.

The sediment loading function is based on the mechanisms of gross erosion and sediment delivery. The Universal Soil Loss Equation (Wischmeier and Smith, 1965) has been chosen to predict the on-site surface (including sheet and rill) erosion, for the following reasons:

1. This equation is applicable to a wide variety of land uses and climatic conditions.
2. It predicts erosion rates by season, in addition to annual averages.
3. Data have been collected nationwide for factors included in the equation.

The sediment loading function has the form:

$$Y(S)_E = \sum_{i=1}^{n} [A_i \cdot (R \cdot K \cdot L \cdot S \cdot C \cdot P \cdot S_d)_i] \qquad \text{(III-1)}$$

where

$Y(S)_E$ = sediment loading from surface erosion, tons/year

n = number of subareas in the area

Source areal factor:

A_i = acreage of subarea i, acres

Source characteristic factors:

R = the rainfall factor, expressing the erosion potential of average annual rainfall in the locality, is a summation of the individual storm products of the kinetic energy of rainfall, in hundreds of foot-tons per acre, and the maximum 30-min. rainfall intensity, in inches per hour, for all significant storms, on an average annual basis

K = the soil-erodibility factor, commonly expressed in tons per acre per R unit

L = the slope-length factor, dimensionless ratio

S = the slope-steepness factor, dimensionless ratio

C = the cover factor, dimensionless ratio

P = the erosion control practice factor, dimensionless ratio

S_d = the sediment delivery ratio, dimensionless

The R factor in the above equation can be expressed in metric units((hundreds of meter-metric tons/ha-cm) multiplied by (maximum 30-min. intensity, cm/hr)) by multiplying the English R values by 1.735. The factor for conversion of K in English units to metric-tons per hectare per metric R unit is 1.292 (Wischmeier, 1972).

Equation III-1 can be used to predict sediment loading resulting from sheet and rill erosion from cultivated and non-cultivated lands. Parameter values for silviculture, construction, and mining are less well documented than for agriculture, however. The user will thus find it relatively easy to use Equation III-1 for agriculture, and substantially more difficult for other sources. The equation does not predict sediment contributions from gully erosion, streambank erosion, or mass soil movement.

The sediment loading section will present the following information:

- procedures and examples for estimating sediment loadings based on the above described loading function
- methodology for predicting minimum and maximum erosion rates.

3.2.1 General Procedure for Use of the Sediment Loading Function

The following procedure is to be used to calculate sediment loading from an area based on the loading function in Equation III-1. The terminology applies to agricultural lands, but the procedure is applicable to other non-point sources. This procedure is shown as a flow diagram in Figure III-1 and is summarized in Sections 3.2.1.1 through 3.2.1.3. Then a more detailed discussion of the procedure follows with example problems being presented in Section 3.2.4.

Estimation of surface erosion should be made for each land-use type. For a given land-use type, if 90% or more of the area is made up of one soil type, one may calculate soil loss for the land use based on that soil type. If there is less than 90% of one soil type, one should calculate soil loss for each soil type that makes up at least 10% of the land use, and then obtain a weighted average for the entire land-use area (U.S. Dept. of Agriculture, 1974).

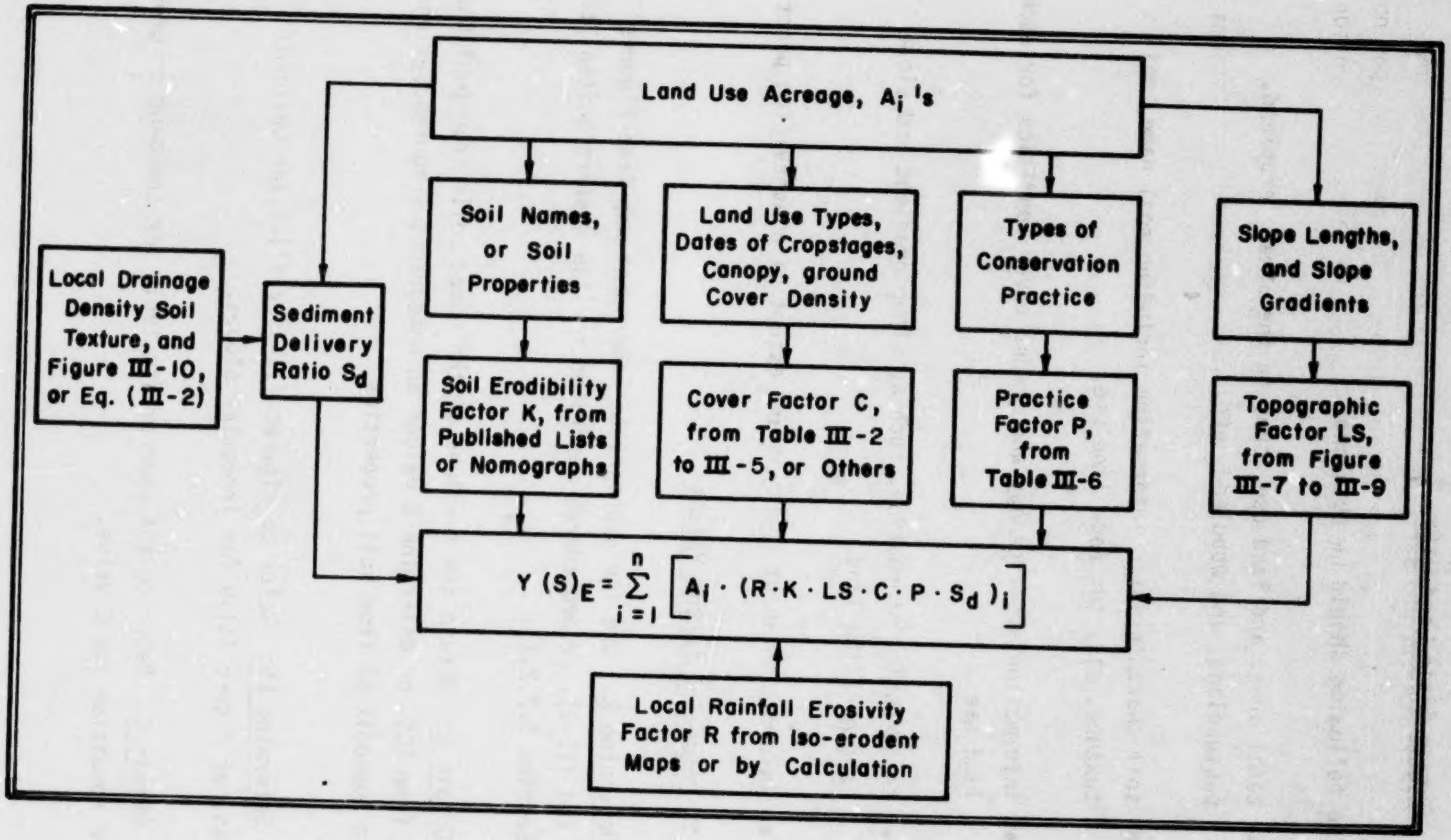

Land Use Acreage, A_i's
Local Drainage Density Soil Texture, and Figure III-10, or Eq. (III-2)
Sediment Delivery Ratio S_d
Soil Names, or Soil Properties
Soil Erodibility Factor K, from Published Lists or Nomographs
Land Use Types, Dates of Cropstages, Canopy, ground Cover Density
Cover Factor C, from Table III-2 to III-5, or Others
Types of Conservation Practice
Practice Factor P, from Table III-6
Slope Lengths, and Slope Gradients
Topographic Factor LS, from Figure III-7 to III-9
$Y(S)_E = \sum_{i=1}^{n} \left[A_i \cdot (R \cdot K \cdot LS \cdot C \cdot P \cdot S_d)_i \right]$
Local Rainfall Erosivity Factor R from Iso-erodent Maps or by Calculation

FIGURE III-1 FLOW DIAGRAM FOR CALCULATING SEDIMENT LOADING FROM SURFACE EROSION

3.2.1.1 Obtain Basic Land Data

The following should be obtained:

- total area, and land use acres in the area: cropland, pastureland, and woodland, etc.
- soil characteristic information including soil name, soil texture, etc. for each land use.
- information about canopy and ground cover condition for each land use.
- topographic information, such as slope gradient and slope length of the land.
- information about the type and extent of conservation practices.

3.2.1.2 Determine Factor Values

Determine R: Use the appropriate isoerodent map (see Figures III-2 and III-3), or procedures described for the western United States (see Section 3.2.2.1).

Obtain K: Obtain the K values of the named soils from published lists from SCS, or determine K values on nomographs (Figures B-1 and B-2 in Appendix B) from soil properties.

Determine LS: Refer to Figures III-7 or III-8 for uniform slopes, or Figure III-9 for irregular slopes.

Obtain C: Refer to the appropriate table for the crop or ground cover condition for C value.

Obtain P: Refer to Table III-6.

Determine sediment delivery ratio, S_d: Obtain from local sources or from Figure III-10 by using drainage density and soil texture for homogeneous watersheds.

3.2.1.3 Calculations

Multiply R, K, LS, C, and P, and S_d to obtain sediment loadings for cropland, pasture, and woodland in annual yields per unit area of source.

Multiply loading rates by source sizes (total hectares or acres) for cropland, pastureland, and woodland to obtain total loading per source.

Sum source loadings calculated in the item above to obtain total loading from land uses (total loading in the watershed will require summation of other sources within the watershed).

3.2.2 Determination of Source Characteristic Factors

3.2.2.1 The Rainfall Factor (R)

R is a factor expressing the erosion potential of precipitation in a locality. It is also called index of erosivity, erosion index, etc. It is the summation of the individual storm products of the kinetic energy of rainfall (denoted by E), and the maximum 30 min. rainfall intensity (denoted by I) for all significant storms within the period under consideration. The product EI reflects the combined potential of raindrop impact and runoff turbulence to transport dislodged soil particles from the site (Wischmeier and Smith, 1965).

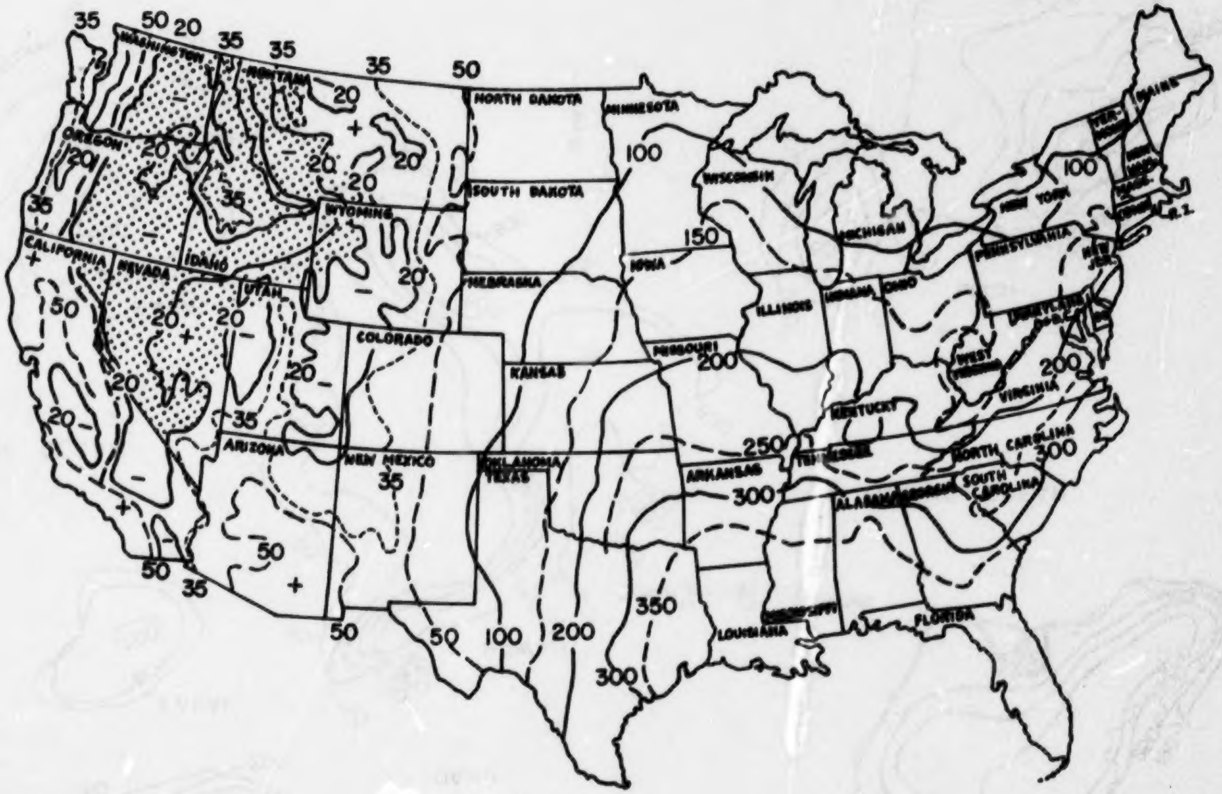
Values of average annual rainfall-erosivity index, R, are shown in Figure III-2 for the continental U.S. and Figure III-3 for islands of Hawaii. On these maps, the lines joining points with the same erosion index value are called isoerodents. Points lying between the indicated isoerodents may be approximated by linear interpolation.

Interpolation for values of R factors in the mountainous areas, particularly those west of the 104th meridian may not be appropriate because of the sporadic rainfall pattern. Values of the erosion index at specific areas can be computed from local recording rain gage records with the help of a rainfall-energy table and the computation procedure presented by Wischmeier and Smith (1958).

ARS recently recommended that 350 be the maximum used in the Gulf and southeastern states, shown in Figure III-2, until further research can validate values higher than 350.

In the northwestern United States, runoff from snowmelt contributes significantly to surface erosion. The annual index of R for some portions of this region is the combined effect of rainfall and snowmelt designated by R_r and R_s, respectively. The snowmelt factor (R_s) is important in Areas A-1, B-1, and C on Figure III-4 (also refer to Table III-1). The map values in the shaded region of the Northwest (see Figure III-2) represent values for the rainfall effect (R_r) only, and must be added with appropriate R_s values to account for the effect of runoff from thaw and snowmelt.

Interim procedures for calculating annual R values, which include both R_r and R_s, for the northwestern U.S. are described in Conservation Agronomy Technical Note No. 32, USDA/SCS, Portland, Oregon (1974), and are briefly presented below.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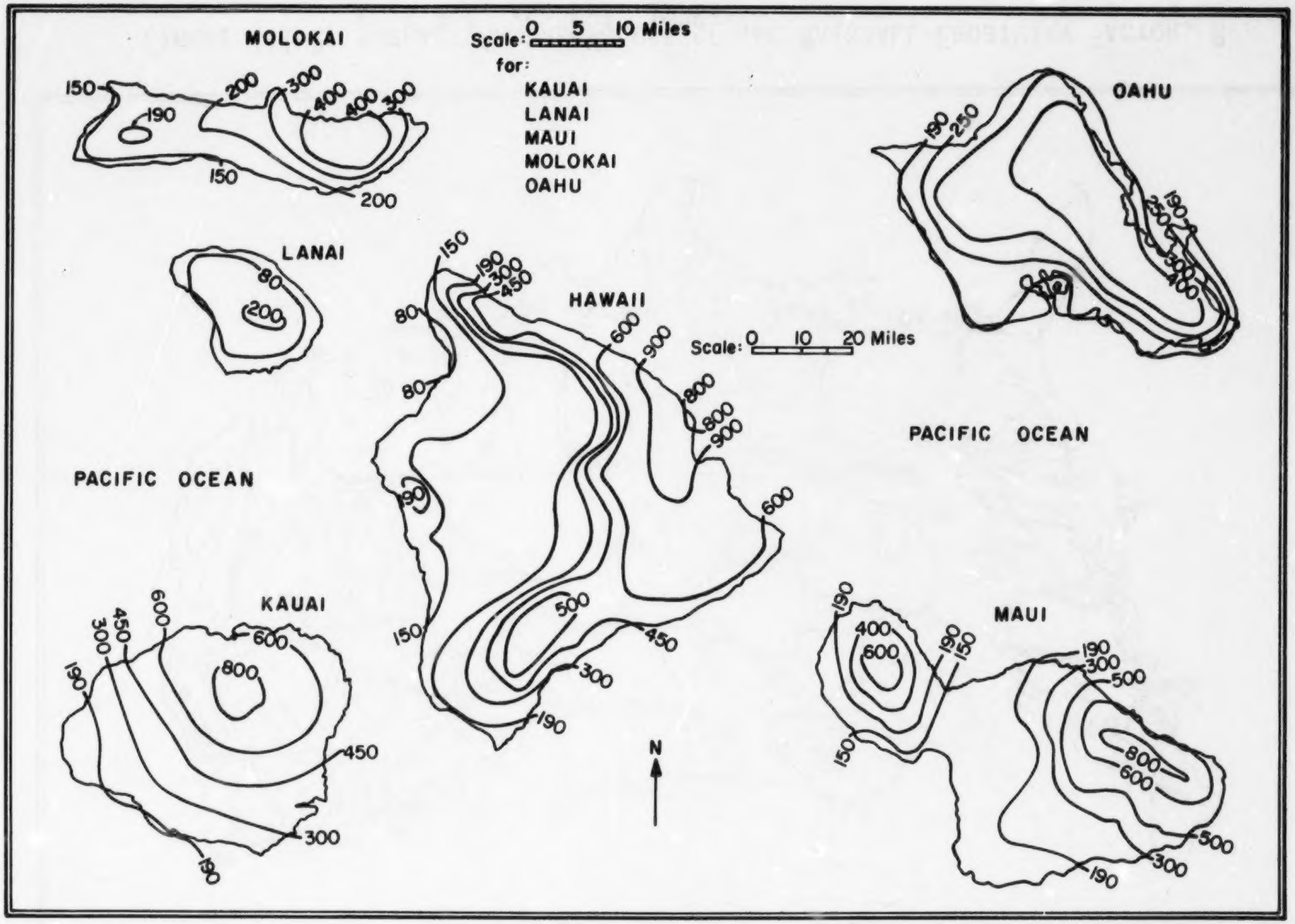
WASHINGTON
OREGON
CALIFORNIA
NEVADA
IDAHO
MONTANA
WYOMING
UTAH
ARIZONA
COLORADO
NEW MEXICO
NORTH DAKOTA
SOUTH DAKOTA
NEBRASKA
KANSAS
OKLAHOMA
TEXAS
MINNESOTA
IOWA
MISSOURI
ARKANSAS
LOUISIANA
WISCONSIN
ILLINOIS
MICHIGAN
INDIANA
OHIO
KENTUCKY
TENNESSEE
MISSISSIPPI
ALABAMA
GEORGIA
FLORIDA
SOUTH CAROLINA
NORTH CAROLINA
VIRGINIA
WEST VIRGINIA
PENNSYLVANIA
NEW YORK
NEW JERSEY
MAINE
R.I.

FIGURE III-2 AVERAGE ANNUAL VALUES OF THE RAINFALL-EROSIVITY FACTOR, R (U.S.D.A. AND EPA, 1975)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MOLOKAI
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LANAI
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Scale: 0 10 20 Miles
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600
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800
800
900
80
80
190
600
150
500
450
300
190
190
250
190
250
300
400
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800
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190
400
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150
150
190
300
500
800
600
500
300
190

Figure III-3 Mean Annual Values of Erosion Index for Hawaii (U.S.D.A., 1974)

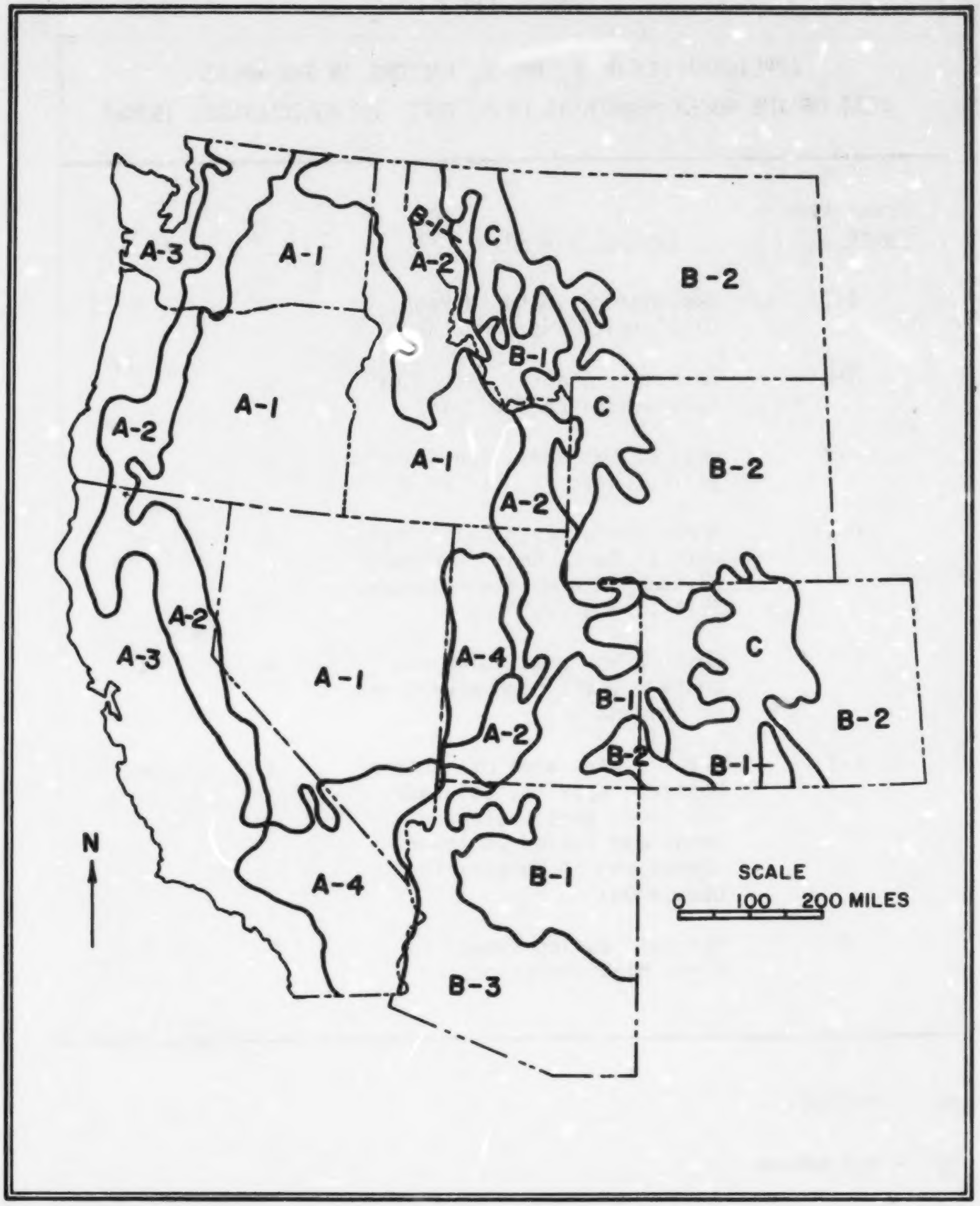

A-3
A-1
B-1
A-2
C
B-2
B-1
C
A-2
A-1
A-1
B-2
A-2
A-2
A-3
A-1
A-4
C
B-1
A-2
B-2
B-2
B-1
N
A-4
B-1
SCALE
0 100 200 MILES
B-3

FIGURE III-4 SOIL MOISTURE-SOIL TEMPERATURE REGIMES OF THE WESTERN UNITED STATES (U.S.D.A., 1974)

TABLE III-1

APPLICABILITY OF R_r AND R_s FACTORS IN THE AREAS WEST OF THE ROCKY MOUNTAINS (U.S. DEPT. OF AGRICULTURE, 1974a)

Areas (see Figure III-4)	Typical Locations	R_r	R_s
A-1	Washington, Idaho, Nevada, California, western Utah	X[a/]	X
A-2	Cascades, Sierra, Tetons of Idaho, Wasatch Mountains	X	-[b/]
A-3	West of Cascades, San Joaquin Valley, west of Sierras	X	-
A-4	Areas of southern California, east of Santa Anas, southern Nevada, intermountain Nevada, Salt Lake area, Utah	X	-
B-1	Western Montana, Colorado, eastern Utah, high elevations of Arizona	X	X
B-2	Great plains area of eastern Montana, Wyoming, Colorado (includes gently sloping mesas and upland at lower elevations of Monticello, Utah area)	X	-
C	Rainfall during summer is high; high elevations	X	X

a/ X needed

b/ - not needed

The annual R_r factor is obtained by using as base the two year, six hour rainfall (2-6 rainfall). Relationships between R_r and 2-6 rainfall vary to conform to specific local climatic characteristics. These relationships are designated as Type I, Type IA, and Type II, and are shown in Figure III-5. Specific areas applicable to these curves are shown in Figure III-6. Type I curve is for the central valley and coastal mountains and valleys of southern California. Type IA curve applies to the coastal side of the Cascades in Oregon and Washington, the coastal side of the Sierra Nevada Mountains in northern California, and the coastal regions of Alaska. Type II curve applies to the remainder of the region. For 2-6 rainfall data, refer to Technical Paper No. 40, U.S. Department of Commerce, Weather Bureau, Washington, D.C. (1961), or other suitable rainfall frequency analysis reports.

To obtain the annual R_s factor for a given location, obtain the average annual total precipitation by snowfall (in inches of water depth) and multiply it by the constant 1.5 to give annual R_s.

There are numerous sources of snowfall data for the United States. Some of the major sources are:

- The 1941 Yearbook of Agriculture, USDA, Washington, D.C.,
- "Climates of the States," Water Information Center, Inc., Port Washington, New York (1974), and
- Data resulting from the Western Federal-State-Private Cooperative Snow Surveys, coordinated by SCS/USDA, Portland, Oregon.

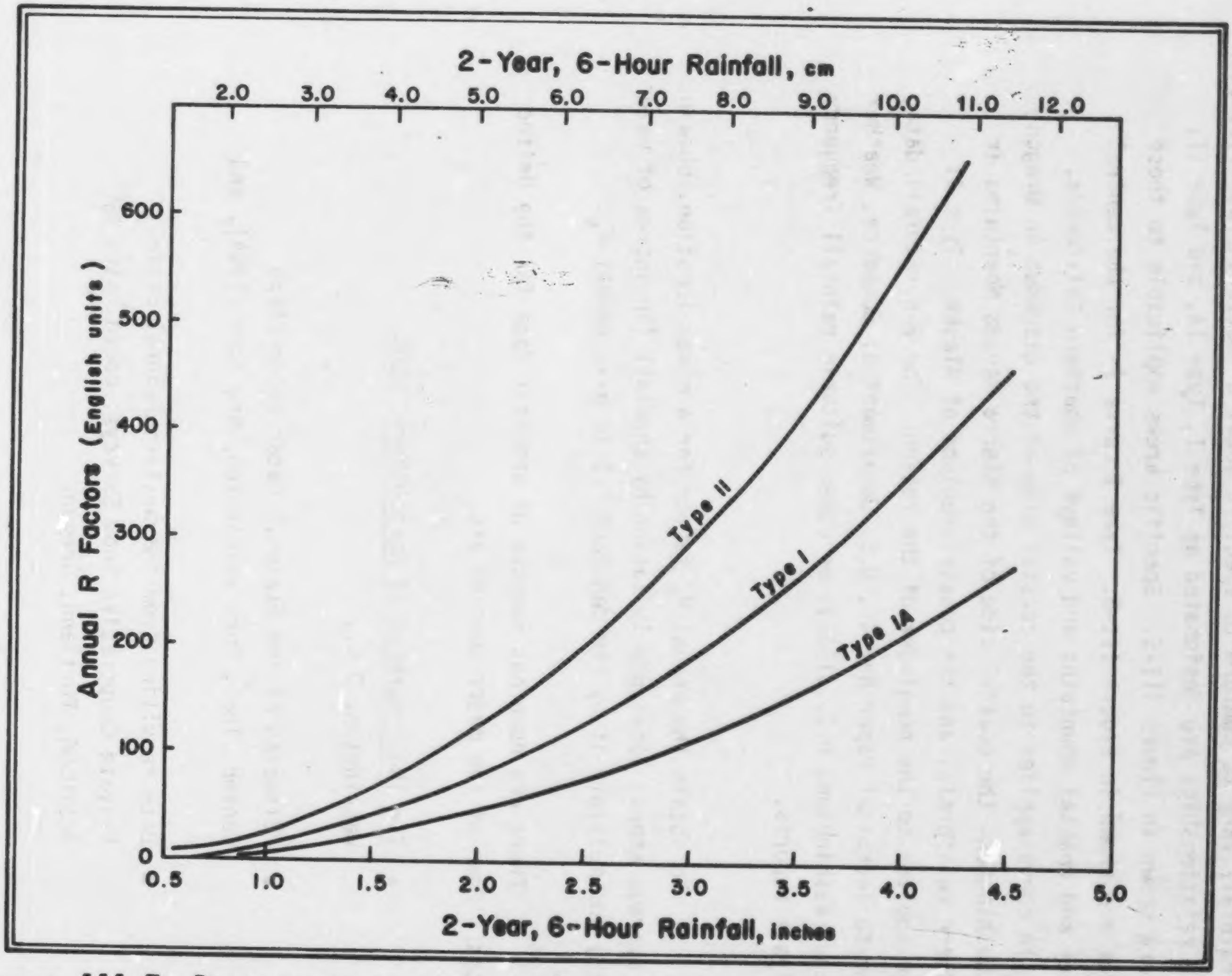


FIGURE III-5 RELATIONSHIPS BETWEEN ANNUAL AVERAGE RAINFALL EROSIVITY INDEX AND THE 2-YEAR, 6 HOUR RAINFALL DEPTH FOR 3 RAINFALL TYPES IN WESTERN U.S. (U.S.D.A., 1974)

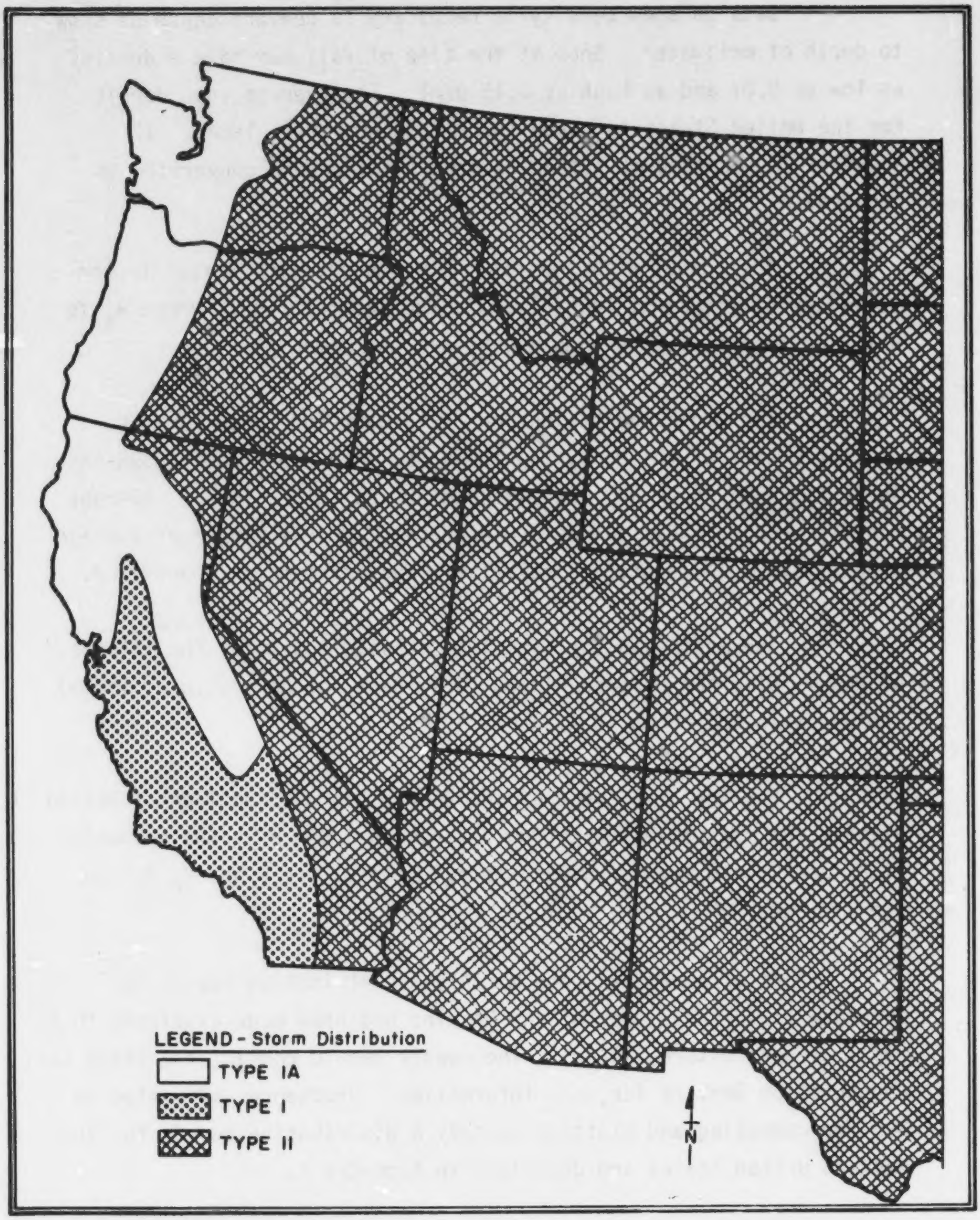

LEGEND - Storm Distribution
TYPE IA
TYPE I
TYPE II
N

FIGURE III-6 STORM DISTRIBUTION REGIONS IN WESTERN U.S. (U.S.D.A., 1974)

Data on snow density is necessary to convert depth of snow to depth of meltwater. Snow at the time of fall may have a density as low as 0.01 and as high as 0.15 g/ml. The average snow density for the United States is taken to be 0.10 (Garstka, 1964). If snowfall is recorded as inches of precipitation, no conversion is required.

The annual R factor for the western United States is the summation of effect of rainfall, R_r, and snowmelt, R_s. Where R_s is not significant, values of R and R_r are the same.

The monthly distribution of the erosion index for the 37 states east of the Rocky Mountains has been reported in USDA-ARS Agriculture Handbook No. 282 (Wischmeier and Smith, 1965). Average monthly erosion index values are expressed as percentages of average annual values and plotted cumulatively against time in Appendix A.

The monthly distribution of erosion index for the islands of Hawaii also has been developed (U.S. Dept. of Agriculture, 1974b). These curves are shown in Appendix A.

For the areas west of the Rockies in the continental United States, the monthly distribution of erosion index R is the summation of R_r and R_s. Where R_s values are not needed, the R and R_r curves are the same.

As of June 1974, the monthly R distribution curves for portions of the area west of the Rockies had been made available (U.S. Dept. of Agriculture, 1974a). The reader should contact the state Soil Conservation Service for such information. Procedures suggested by SCS for computing and plotting monthly R distribution curves for the western United States are described in Appendix A.

3.2.2.2 The Soil-Erodibility Factor (K)

K factor is a quantitative measure of the rate at which a soil will erode, expressed as the soil loss (tons) per acre per unit of R, for a plot with 9% slope, 72.6 ft. long, under continuous cultivated fallow.

K factors for topsoils, as well as subsoils, for most soil series have been developed. Values of K for soils studied thus far vary from 0.12 to 0.70 tons/acre/unit R.

The K values for named soils at different locations of the nation can be obtained from the regional or state offices of the Soil Conservation Service.

K values of soils can be predicted from soil properties. In Appendix B of this report, two nomographs are presented from which K values may be determined for topsoils and subsoils when the governing soil properties are known.

3.2.2.3 The Topographic Factor (LS)

Soil loss is affected by both length (L) and steepness of slope (S). These factors affect the capability of runoff to detach and transport soil material.

The slope length factor is the ratio of soil loss from a specific length of slope to that length (72.6 ft) specified for the K factor in the equation. Slope length is defined as the distance from the point of origin of overland flow to either of the following, whichever is limiting, for the major part of the area under consideration: the point where the slope decreases to the extent that deposition begins; or the point where runoff enters a well-defined channel that may be part of a drainage network, or a constructed channel that may be part of a drainage network, or a constructed channel such as a terrace or diversion. Slope length can be determined accurately by on-site inspection of a field, or by measurements from aerial photographs, or topographic maps. When the land is terraced, the terrace spacing should be used. All slope lengths are compared to a slope length of 72.6 ft, which has a factor value of 1.

The slope gradient or percent slope factor is the ratio of soil loss from a specific percent slope to that slope (9%) specified for the K value in the USLE. A 9% slope has a factor value of 1. Slope data may be obtained from topographic maps, engineering or land level surveys, and other sources. A widely used method is to estimate slope from soil survey maps in which the soils have been mapped by slope range.

The slope length (L) and slope gradient (S) are combined in the USLE into a single dimensionless topographic factor, LS, which can be evaluated using a slope-effect chart.

The slope-effect chart in Figure III-7 is designed for the following areas shown in Figure III-4: A-1 in Washington, Oregon, and Idaho; and all of A-3 (U.S. Dept. of Agriculture, 1974a).

For the remainder of the U.S., the slope-effect chart, Figure III-8, is to be used (U.S. Dept. of Agriculture, 1974a).

Slope-effect charts in Figure III-7 and III-8 can be used when uniform slopes are assumed. The following steps are to be used for obtaining LS values from these charts:

1. Enter the chart on the horizontal axis with the appropriate value of slope length.

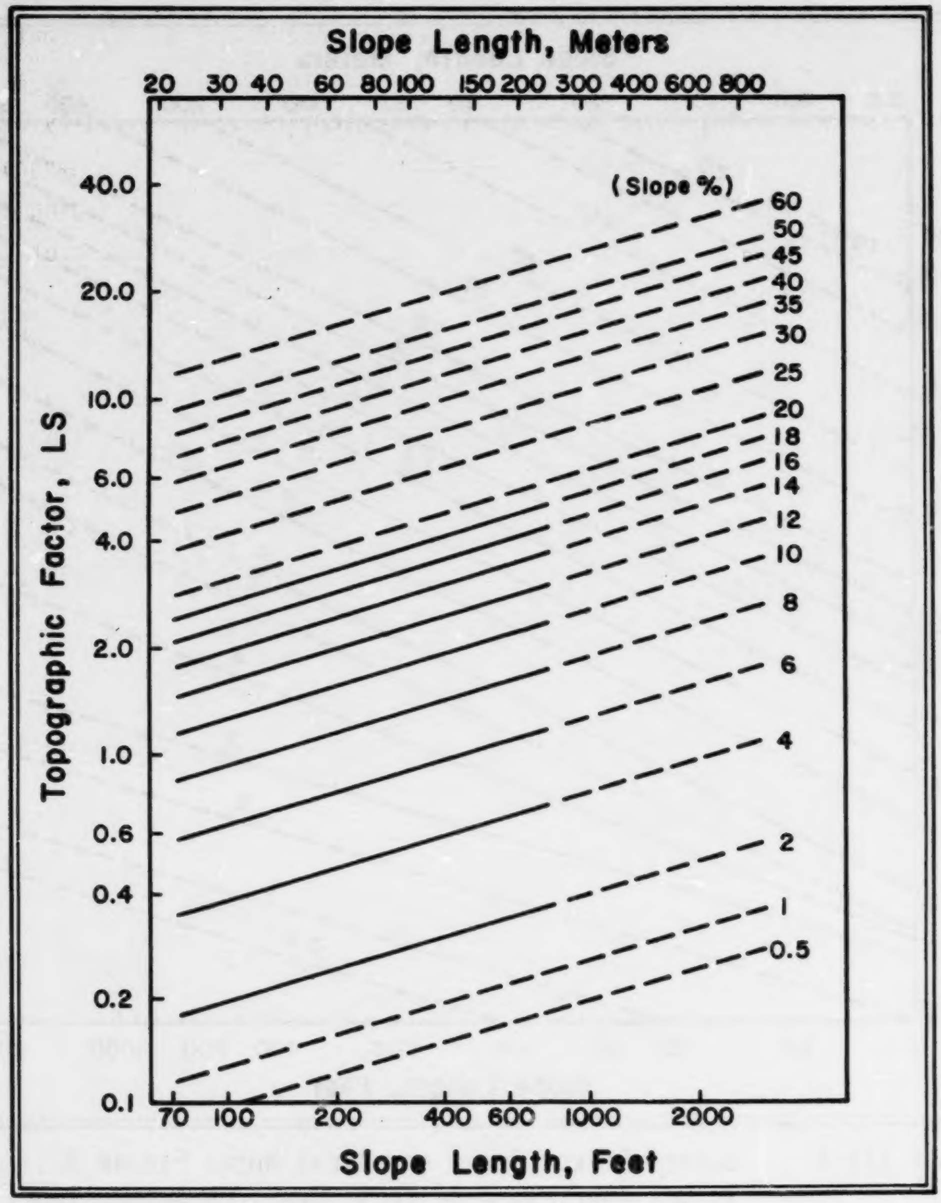


FIGURE III-7 SLOPE EFFECT CHART APPLICABLE TO AREAS A-1 IN WASHINGTON, OREGON, AND IDAHO AND ALL OF A-3 A,B/ (U.S. DEPT. OF AGRICULTURE, 1974)

A/ SEE FIGURE III-4.

B/ DASHED LINES ARE EXTENSIONS OF LS FORMULAE BEYOND VALUES TESTED IN STUDIES.

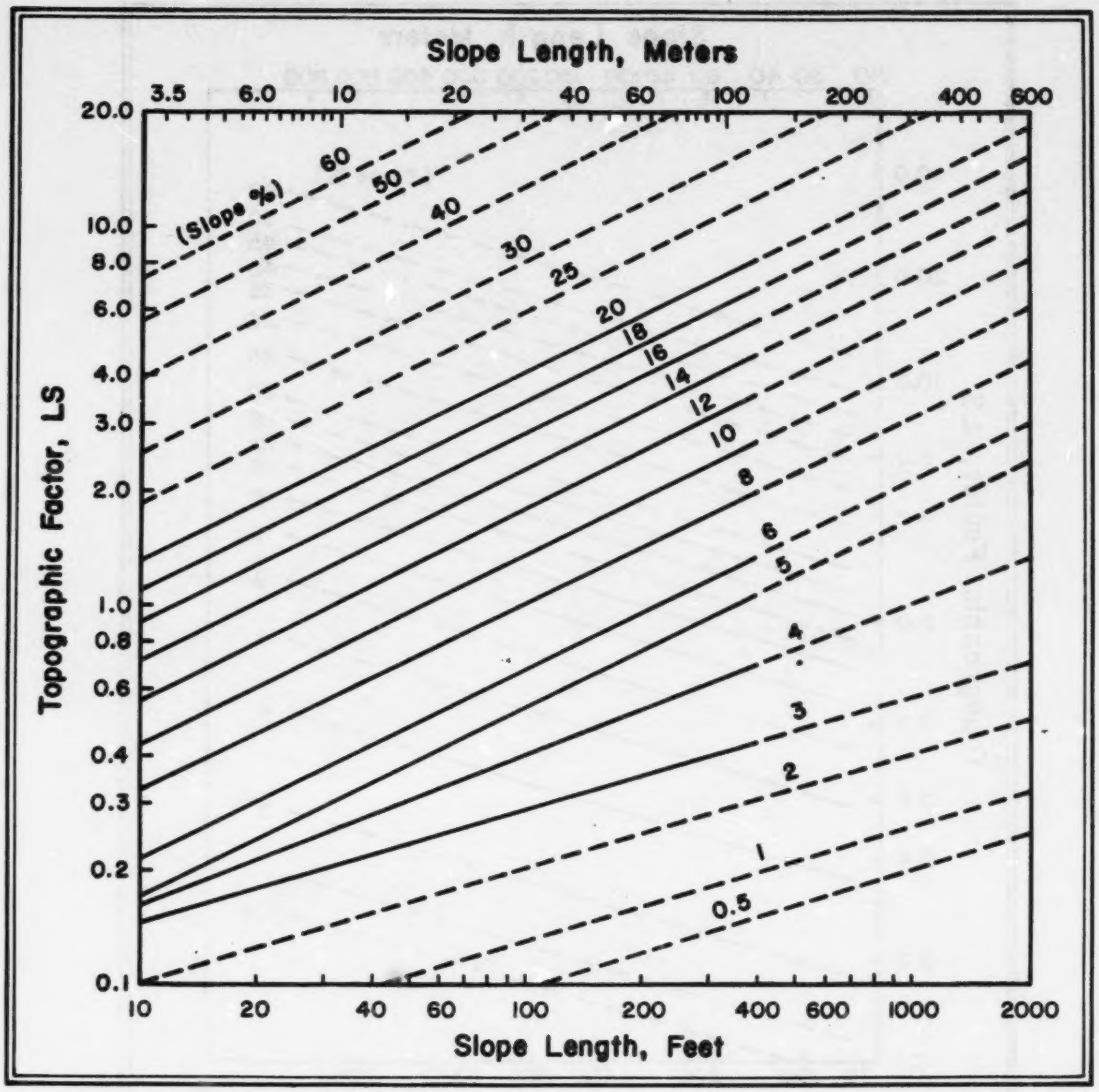


FIGURE III-8 SLOPE--EFFECT CHART FOR AREAS WHERE FIGURE III-7 IS NOT APPLICABLE A/ (U.S. DEPT. OF AGRICULTURE)

A/ THE DASHED LINES REPRESENT ESTIMATES FOR SLOPE DIMENSIONS BEYOND THE RANGE OF LENGTHS AND STEEPNESSES FOR WHICH DATA ARE AVAILABLE.

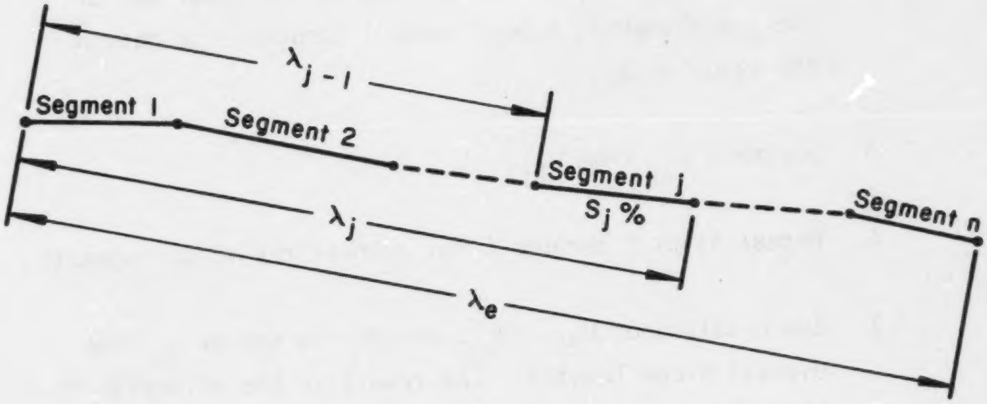
2. Follow the vertical line for that slope length to where it intersects the curve for the appropriate percent slope.

3. Read across the point of intersection to the vertical axis. The number on the vertical axis is the LS value.

An irregular slope should be divided into a series of segments such that the slope gradient within each segment can be treated as uniform. The slope segments need not be of equal length. The total soil loss from the entire slope is calculated based on the effective LS value for the entire length of the irregular slope.

A family of curves shown in Figure III-9 was designed to facilitate the determination of the LS factor for the irregular slopes ranging from 2 to 20%. The quantity plotted on the vertical scale is designated by the symbol U. Slope lengths, designated by λ, are plotted on the horizontal scale.

Assume an irregular slope with n segments illustrated as follows:



where

λ_j = distance from the top of the entire slope (the point at which overland flow begins) to the lower end of the jth segment

λ_{j-1} = length of entire slope above segment j

λ_e = overall slope length

S_j = the slope gradient of segment j, in percent

The steps taken for calculating LS for irregular slopes using Figure III-9 are (Foster and Wischmeier, 1973):

1. Enter on the horizontal axis with the value of λ_{j-1} (the slope length above segment j).

2. Move vertically to the curve with the appropriate percent slope for segment j.

3. Read on the vertical scale the value of U_{1j}.

4. Enter the figure with the value of λ_j (the distance from the top of the entire slope to the lower end of the jth segment), repeat Steps 1 through 3 to obtain the value of U_{2j}.

5. Subtract U_{2j} from U_{1j}.

6. Repeat Steps 1 through 5 for each of the slope segments.

7. Sum n values of $U_{2j} - U_{1j}$, divide the sum by λ_e (the overall slope length). The result is the effective LS value for the entire length of the irregular slope.

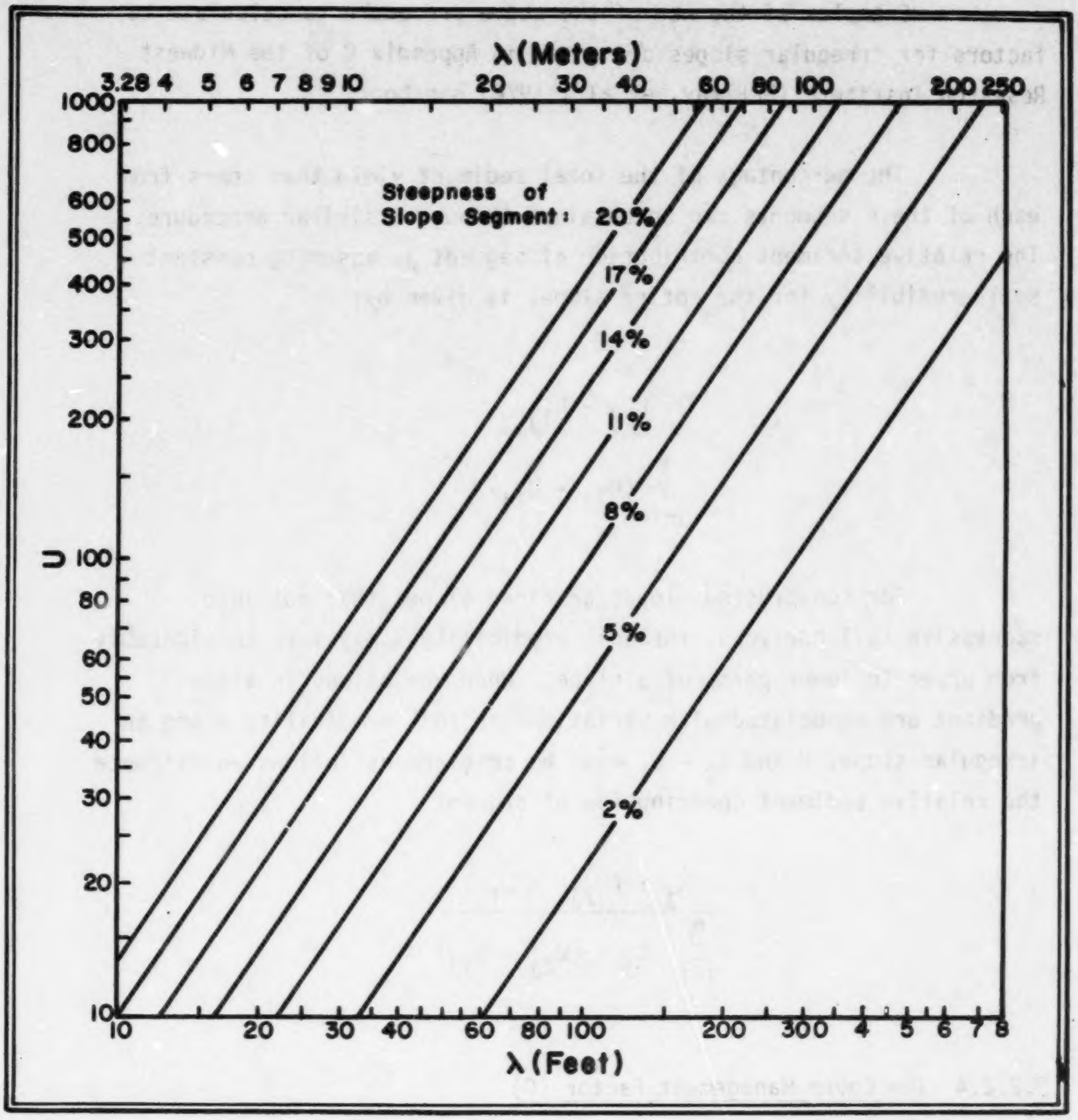


FIGURE III-9 SLOPE EFFECT CHART FOR IRREGULAR SLOPES (FORTER AND WISCHMEIER, 1973)

Examples of the use of the above procedure to calculate LS factors for irregular slopes are given in Appendix C of the Midwest Research Institute (McElroy, et al., 1976) handbook.

The percentage of the total sediment yield that comes from each of the n segments can be obtained through a similar procedure. The relative sediment contribution of segment j, assuming constant soil erodibility for the entire slope, is given by:

$$\frac{U_{2j} - U_{1j}}{\sum_{j=1}^{n} (U_{2j} - U_{1j})}$$

For constructed slopes or mined slopes that cut into successive soil horizons, the soil erodibility K may vary considerably from upper to lower parts of a slope. When variations in slope gradient are associated with variations in soil erodibility along an irregular slope, K and $U_2 - U_1$ must be combined as follows to estimate the relative sediment contribution of segment j.

$$\frac{K_j \cdot (U_{2j} - U_{1j})}{\sum_{j=1}^{n} K_j \cdot (U_{2j} - U_{1j})}$$

3.2.2.4 The Cover Management Factor (C)

In the USLE, the factor C represents the ratio of soil quantity eroded from land that is cropped or treated under a specified condition to that which is eroded from clean-tilled fallow under identical slope and rainfall conditions. C ranges in value from near zero for excellent sod or a well-developed forest to 1.0 for continuous fallow, construction areas, or other extensively disturbed soil.

In order to evaluate the cover management factor for crops, five crop stage periods have been selected for relative uniformity of cover and residue effects within each period. These five periods are defined as follows (Wischmeier and Smith, 1965):

Period F: Rough fallow - turn plowing to seeding.

Period 1: Seedbed - seeding to 1 month thereafter.

Period 2: Establishment - from 1 to 2 months after seeding. (Exception: for fall-seeded grain, Period 2 includes the winter period and extends to 30 April in the North and 1 April in the South, with intermediate latitudes interpolated.)

Period 3: Growing crop - from Period 2 to crop harvest.

Period 4: Harvest, residue or stubble - from crop harvest to turn plow or new seedbed. (When meadow is established in small grain, Period 4 ends 2 months after grain harvest. Thereafter, it is classified as established meadow.)

The average cover factor C for the entire year or years of crop rotation is computed by crop stages. Input for calculation of C includes average planting and harvesting dates, productivity, disposition of crop residues, tillage, and monthly distribution of the erosion index R. The C value for each of these time periods is weighted according to the percentage of annual rainfall factor occurring in that period. The summation of these RC products for the entire year or years of crop rotation is then converted to a mean annual C.

Values of factor C for croplands are highly variable with rainfall pattern, planting dates, type of vegetative cover, seeding method, soil tillage, disposition of residues, and general management level. Ranges of C value for several types of vegetation and ground cover are listed in Table III-2, in order of decreasing protection against erosion (increasing C value from near zero to 1).

The reader is advised to consult with state conservation agronomists of SCS for appropriate C values for crops in the local area. The reader is also referred to USDA-ARS Agriculture Handbook No. 282 (Wischmeier and Smith, 1965) for a listing of approximated C values for various crops at each crop stage, as well as a working table for derivation of average C value for periods of crop rotation.

C values typical of permanent pasture, range, and idle lands, with varying cover and canopy conditions, are given in Table III-3. These values were developed by Wischmeier (1972).

Wischmeier (1972) also estimated factor C for some woodland situations. Data are presented in Table III-4.

For urban and road areas, as well as construction sites, the factor C represents the effect of land cover or treatment that may be used to protect soil from being eroded. Table III-5 (Water Resources Administration, 1973) lists values of the factor C for various soil covers and treatments.

3.2.2.5 The Practice Factor (P)

The factor P accounts for control practices that reduce the erosion potential of runoff by their influence on drainage patterns, runoff concentration, and runoff velocity.

TABLE III-2

RELATIVE PROTECTION OF GROUND COVER AGAINST EROSION (IN ORDER OF INCREASING C VALUES)

Land-Use Groups	Examples	Range of "C" Values
Permanent vegetation	Protected woodland	0.0001-0.45
	Prairie	
	Permanent pasture	
	Sodded orchard	
	Permanent meadow	
Established meadows	Alfalfa	0.004-0.3
	Clover	
	Fescue	
Small grains	Rye	0.07-0.5
	Wheat	
	Barley	
	Oats	
Large-seeded legumes	Soybeans	0.1-0.65
	Cowpeas	
	Peanuts	
	Field peas	
Row crops	Cotton	0.1-0.70
	Potatoes	
	Tobacco	
	Vegetables	
	Corn	
	Sorghum	
Fallow	Summer fallow	1.0
	Period between plowing and growth of crop	

TABLE III-3

"C" VALUES FOR PERMANENT PASTURE, RANGELAND, AND IDLE LAND (WISCHMEIER, 1972)[a]

Vegetal Canopy	Canopy		Cover that Contacts the Surface					
			Percent Ground Cover					
Type and Height of Raised Canopy[b]	Cover[c] (%)	Type[d]	0	20	40	60	80	95-100
Column No.	2	3	4	5	6	7	8	9
No appreciable canopy		G	0.45	0.20	0.10	0.042	0.013	0.003
		W	0.45	0.24	0.15	0.090	0.043	0.011
Canopy of tall weeds	25	G	0.36	0.17	0.09	0.038	0.012	0.003
or short brush		W	0.36	0.20	0.13	0.082	0.041	0.011
(0.5 m fall height)	50	G	0.26	0.13	0.07	0.035	0.012	0.003
		W	0.26	0.16	0.11	0.075	0.039	0.011
	75	G	0.17	0.10	0.06	0.031	0.011	0.003
		W	0.17	0.12	0.09	0.067	0.038	0.011
Appreciable brush	25	G	0.40	0.18	0.09	0.040	0.013	0.003
or brushes		W	0.40	0.22	0.14	0.085	0.042	0.011
(2 m fall height)	50	G	0.34	0.16	0.085	0.038	0.012	0.003
		W	0.34	0.19	0.13	0.081	0.041	0.011
	75	G	0.28	0.14	0.08	0.036	0.012	0.003
		W	0.28	0.17	0.12	0.077	0.040	0.011

(continued)

TABLE III-3 (Continued)

Vegetal Canopy Type and Height of Raised Canopy b/	Canopy Cover c/ (%)	Type d/	Cover that Contacts the Surface					
			Percent Ground Cover					
			0	20	40	60	80	95-100
Column No.	2	3	4	5	6	7	8	9
Trees but no appreciable low brush (4 m fall height)	25	G	0.42	0.19	0.10	0.041	0.013	0.003
		W	0.42	0.23	0.14	0.087	0.042	0.011
	50	G	0.39	0.18	0.09	0.040	0.013	0.003
		W	0.39	0.21	0.14	0.085	0.042	0.011
	75	G	0.36	0.17	0.09	0.039	0.012	0.003
		W	0.36	0.20	0.13	0.083	0.041	0.011

---

a/ All values shown assume: (1) random distribution of mulch or vegetation, and (2) mulch of appreciable depth where it exists.

b/ Average fall height of waterdrops from canopy to soil surface: m = meters.

c/ Portion of total-area surface that would be hidden from view by canopy in a vertical projection (a bird's-eye view).

d/ G: Cover at surface is grass, grasslike plants, decaying compacted duff, or litter at least 5 cm (2 in.) deep.

W: Cover at surface is mostly broadleaf herbaceous plants (as weeds) with little lateral-root network near the surface and/or undecayed residue.

TABLE III-4

"C" VALUES FOR WOODLAND (WISCHMEIER, 1972)

Stand Condition	Tree Canopy Percent of Area[a]	Forest Litter Percent of Area[b]	Undergrowth[c]	"C" Factor
Well stocked	100-75	100-90	Managed[d]	0.001
			Unmanaged[d]	0.003-0.011
Medium stocked	70-40	85-75	Managed	0.002-0.004
			Unmanaged	0.01-0.04
Poorly stocked	35-20	70-40	Managed	0.003-0.009
			Unmanaged	0.02-0.09[e]

---

[a] When tree canopy is less than 20%, the area will be considered as grassland or cropland for estimating soil loss.

[b] Forest litter is assumed to be at least 2-in. deep over the percent ground surface area covered.

[c] Undergrowth is defined as shrubs, weeds, grasses, vines, etc., on the surface area not protected by forest litter. Usually found under canopy openings.

[d] Managed - grazing and fires are controlled.
Unmanaged - stands that are overgrazed or subjected to repeated burning.

[e] For unmanaged woodland with litter cover of less than 75%, C values should be derived by taking 0.7 of the appropriate values in Table III-3. The factor of 0.7 adjusts for the much higher soil organic matter on permanent woodland.

TABLE III-5

"C" VALUES FOR CONSTRUCTION SITES
(WATER RESOURCES ADMINISTRATION, 1973)

Type of Cover	C Value
None (fallow)	1.00
Temporary seedings	
First 60 days	0.40
After 60 days	0.05
Permanent seedings	
First 60 days	0.40
After 60 days	0.05
After 1 year	0.01
Sod (laid immediately)	0.01

	Rate of Application			Maximum Allowable Slope Length	
Mulch	In Metric Tons Per Hectare	In Tons Per Acre	C Value	(ft)	(m)
Hay or straw	1/2	1/2	0.34	20	6
	1	1	0.20	30	9
	1-1/2	1-1/2	0.10	40	12
	2	2	0.05	50	15
Stone or gravel	14	15	0.80	15	5
	55	60	0.20	80	24
	120	135	0.10	175	53
	220	240	0.05	200	61
Chemical mulches					
First 90 days		a/	0.50	50	15
After 90 days		a/	1.00	50	15
Woodchips	2	2	0.80	25	8
	4	4	0.30	50	15
	6	7	0.20	75	23
	11	12	0.10	100	30
	18	20	0.06	150	46
	23	25	0.05	200	61

a/ As recommended by manufacturer.

For croplands, control practices refer to contour tillage, cross-slope farming, and contour strip-cropping. The practice value P is the ratio of soil loss from a specified conservation practice to the soil loss occurring with up-and-downhill tillage, when other conditions remain constant. Table III-6 (U.S. Dept. of Agriculture, 1974a) shows P values for up-and-downhill farming, cross-slope farming without strips, contour farming, cross-slope farming with strips, and contour strip-cropping.

TABLE III-6

"P" VALUES FOR EROSION CONTROL PRACTICES ON CROPLANDS (U.S. DEPT. OF AGRICULTURE, 1974a)

Slope	Up-and-Down-hill	Cross-slope Farming Without Strips	Contour Farming	Cross-slope Farming with Strips	Contour Strip-cropping
2.0-7	1.0	0.75	0.50	0.37	0.25
7.1-12	1.0	0.80	0.60	0.45	0.30
12.1-18	1.0	0.90	0.80	0.60	0.40
18.1-24	1.0	0.95	0.90	0.67	0.45

Terracing is also an effective practice to reduce soil erosion. The quantitative effect of terracing is accounted for in the slope length factor, since the horizontal terrace interval becomes the slope length, after the terraces are constructed.

3.2.2.6 Sediment Delivery Ratio (S_d)

The sediment-delivery ratio, in this report, is defined as the fraction of the gross erosion which is delivered to a stream. The classical method for determining an average delivery ratio is by comparing the magnitude of the sediment yield at a given point in a watershed (generally at a reservoir or a stream sediment measuring station), and the total amount of erosion. The quantities of gross erosion from sloping uplands are computed by erosion prediction equations for surface erosion, and estimated by various procedures for gullies, stream channels, and other sources. The sediment yield at a given downstream point is obtained through direct measurements.

Measurements of sediment accumulations in reservoirs and sediment-load records in streams show wide variations in sediment yields from watersheds. Estimates show that as little as 5% and as much as 100% of the materials eroded in some watersheds may be delivered to a downstream point. Estimates of the delivery ratio for some specific watersheds, particularly in the humid sections of the country, can be obtained from the Soil Conservation Service, USDA.

Many delivery-ratio analysis studies have been aimed at finding measurable influencing factors that can be related to sediment-delivery ratio. A popular means of developing such information is by statistical analysis using the sediment-delivery ratio as the dependent variable and measurable watershed factors as the independent, or controlling variables. Many physical and hydrologic factors may influence sediment-delivery ratios. Some lend themselves to quantitative expression whereas others do not. At present, the science of sedimentology does not delineate the relative degree of influence of each of the individual factors on the delivery ratio of sediment. Nevertheless, empirical relationships for delivery ratios have been proposed and are presented below. Estimates of sediment loading can be made through the use of these relationships, but such estimates should be tempered with judgment and consideration of other influencing factors not included in the quantitative expressions. The user is encouraged to consult with local experts and should use local data when available.

The MITRE Corporation (1974) reported that the sediment-delivery ratio for construction sites can be approximated by a function of the overland distance between the construction site and the receiving water.

The format of the sediment delivery ratio proposed by MITRE for construction sites has the following form:

$$S_d = D^{-0.22} \tag{III-2}$$

where

D = overland distance between the erosion site and the receiving water, in ft

The above equation was empirically derived from available data. The data base for the derivations includes values of D from 0 to 800 ft. MITRE suggests that this function should be further tested, particularly in areas of the Midwest and Central U.S.

For mining sites, logging roads and fire lanes, sediment delivery ratio relationships have not yet been established due to lack of systematically measured data. It is suggested that the delivery ratio developed by MITRE and expressed in Equation III-2 be used as the first approximation for these sites. This needs to be verified when appropriate data become available.

Sediment delivery ratios have been evaluated in many areas of the country, particularly the eastern half of the United States. The delivery ratio usually depicts a general trend in basins that are relatively homogeneous with respect to soils, land cover, climate, and topography. The Soil Conservation Service (1973) analyzed data from stream and reservoir sediment surveys from widely scattered areas.

This analysis shows that sediment delivery ratios vary inversely with "drainage basin size". It also indicates the effect of soil texture of upland soil on the sediment delivery ratio.

The delivery ratio relationships reported by SCS (1973) were used by the MRI study group in developing delivery ratios for sediment loading to watercourses. The result is shown in Figure III-10. The horizontal scale of the figure is the reciprocal of drainage density which is defined as the ratio of total channel-segment lengths (accumulated for all orders within a basin) to the basin area. The reciprocal of drainage density may be thought of as an expression of the closeness of spacing of channels, or the average distance for soil particles to travel from erosion site to the receiving water.

The delivery ratio relationship shown in Figure III-10 also takes into account the effect of soil texture. For example, if soil texture of upland soil is essentially silt or clay, the sediment delivery ratio will be higher than when the soil texture is coarse.

The delivery ratio relationships in Figure III-10 need to be verified by acquisition of new data and improved by including other factors relative to deposition mechanisms.

The following steps are to be used to obtain the delivery ratio (S_d) from Figure III-10.

1. Enter the figure on the horizontal axis with the value of the reciprocal of drainage density (1/DD).

2. Move vertically from the value of 1/DD to where it intersects the curve for the appropriate soil texture.

3. Read across from the point of intersection to the vertical axis. That number represents the delivery ratio, S_d.

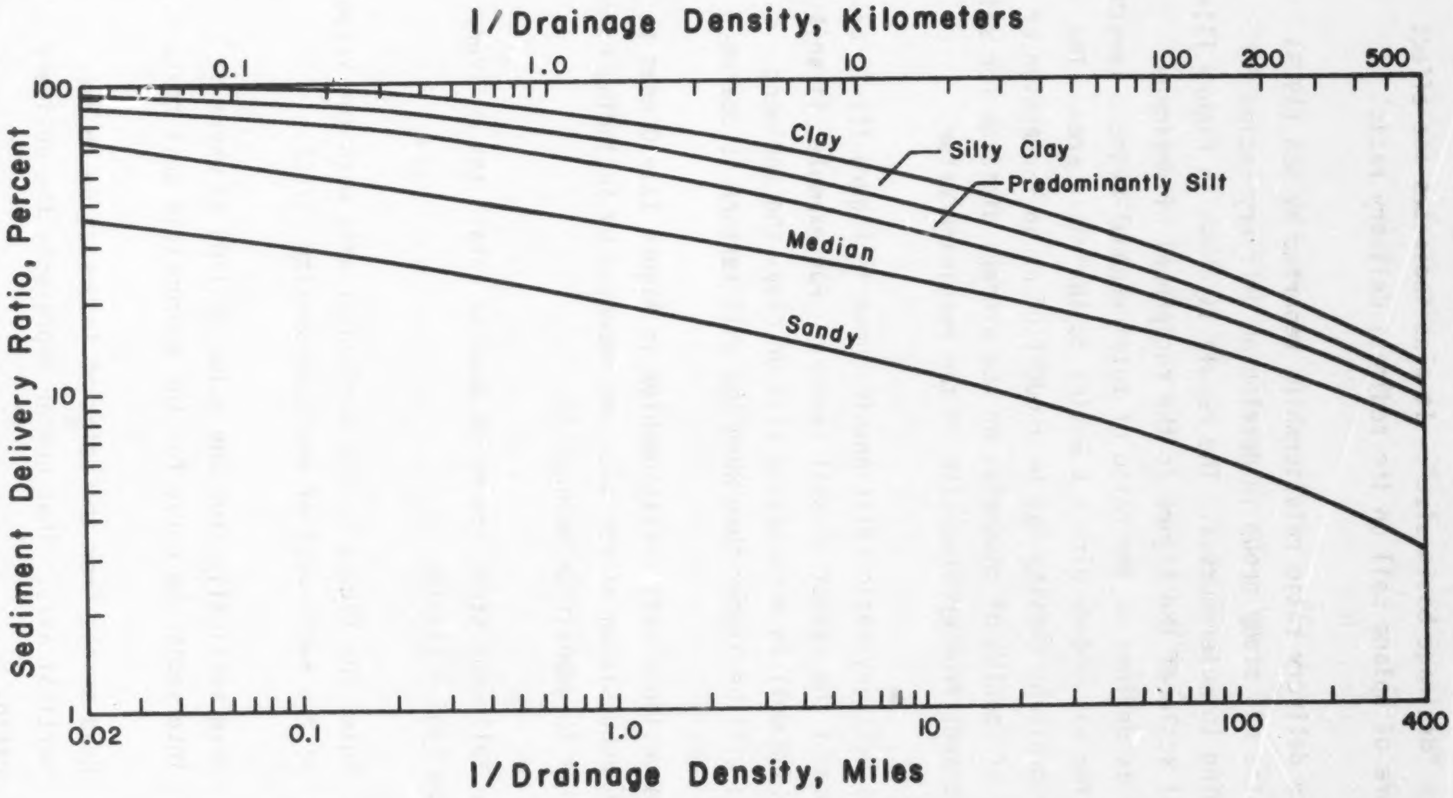


FIGURE III-10 SEDIMENT DELIVERY RATIO FOR RELATIVELY HOMOGENEOUS BASINS (McELROY, ET AL., 1976)

A great range of values of drainage density exists in the United States, from 2 km/km^2 (3 miles/mile^2) for the Appalachian Plateau Province (Smith, 1950) to 500 km/km^2 (800 miles/mile^2) in Badlands at Perth Amboy, New Jersey (Schumm, 1956). In general, according to Strahler (1964), low drainage density is found in regions of highly resistant or highly permeable subsoil materials, under dense vegetative cover, and where relief is low. High drainage density is favored in regions of weak or impermeable materials, sparse vegetation, and mountainous relief.

Some typical values of drainage density for various locales in the U.S. are given in Table III-7. Local drainage density figures may be obtained from agencies such as the U. S. Geological Survey and the Army Corps of Engineers.

Measurements of drainage density can be made from a topographic map with a planimeter and chartometer. Care must be taken to include all permanent stream channels to their upper ends by checking in the field or aerial photographs in verification of topographic maps. A rapid approximation method for determining drainage density is suggested by Carlston and Langbein (1960).

3.2.2.7 Summary of Applicabilities of Source Characteristic Factors

The preceding paragraphs indicate that assessment of sediment loadings from surface erosion requires quantitative information on soil erodibility, rainfall and snowmelt erosivity, topography, vegetative cover, conservation practices, and sediment delivery ratio. Applicability of each factor varies with specific location of the site and also with type of land disturbance. Table III-8 gives a total summary of variations in application of those factors.

TABLE III-7

TYPICAL VALUES OF DRAINAGE DENSITY

Location	Drainage density km/km^2	Drainage density mile/mile^2	Reference
Appalachian Plateau Province	1.9-2.5	3.0-4.0	Smith (1950)
Central and eastern United States	5-10	8.0-16.0	Strahler (1952)
Dry Areas of the Rocky Mountain Region	31-62	50-100	Melton (1957)
The Rocky Mountain Region (except the above)	5-10	8.0-16	Melton (1957)
Coastal ranges of Southern California	12-25	20-40	Smith (1950) Melton (1957) Maxwell (1960)
Badlands in South Dakota	125-250	200-400	Smith (1958)
Badlands in New Jersey	183-510	310-820	Schumn (1956)

3.2.3 Limitations of the Loading Function

The USLE predicts soil losses from sheet and rill erosion. It does not predict sediment from gullies, streambank erosion, landslides, road ditches, irrigation, or from wind erosion. The USLE was developed primarily for croplands, and has been chiefly based upon experimental plot data from the areas east of the Rocky Mountains. The loading function therefore is best defined for these areas of use. For croplands in the western United States and sources outside agriculture such as silviculture, construction, and mining, the factors have not been systematically developed, which seriously affects the ease of using the USLE for such sources.

TABLE III-8

SUMMARY OF APPLICABILITY OF CHARACTERISTIC FACTORS

Land use		Source characteristic factor	Regions in the United States	
Extent of land disturbance	Example		Eastern states and Hawaii	Western states
Zero to moderately disturbed	Growing forests Rangeland Pastureland Cropland Orchards	R	Affected by rainfall only; use Figures III-2 or III-3.	Affected by rainfall, some areas also by snowmelt (see note below).
		K	Erodibility of topsoils.	Erodibility of topsoils.
		LS	Use Figure III-8 for natural slope steepness and slope length (except terracing).	Use Figure III-7 for some areas in Washington, Oregon, Idaho, and California; the remainder use Figure III-8 (except terracing).
		C	For croplands and orchards, C's are determined locally by SCS. For forests, use Table III-4; rangeland and pastureland, use Table III-3.	For croplands and orchards, C's are determined locally by SCS. For forests, use Table III-4; rangeland and pastureland, use Table III-3.
		P	For croplands, use Table III-6; others = 1.0.	For croplands, use Table III-6; others = 1.0.
		S_d	Assume relatively homogeneous land use components; use Figure III-10.	Assume relatively homogeneous land use components; use Figure III-10.

TABLE III-8 (Continued)

Land use		Source characteristic factor	Regions in the United States	
Extent of land disturbance	Example		Eastern states and Hawaii	Western states
Intensively disturbed	Construction sites Mining sites Logging roads Fire lanes	R	Affected by rainfall only; use Figure III-2 or III-3.	Affected by rainfall, some areas also by snowmelt (see note below).
		K	Erodibility of topsoils and subsoils.	Erodibility of topsoils and subsoils.
		LS	Use Figure III-9 for irregular slopes.	Use Figure III-9 for irregular slopes.
		C	Use Table III-5.	Use Table III-5.
		P	Equals 1.0.	Equals 1.0.
		S_d	Use Equation III-2.	Use Equation III-2.

Note: To develop annual R values for the western United States see Section 3.2.2.1.

3.2.3.1 Specific Limitations

R: Research is needed to determine the effective R values more accurately throughout the continental United States.

L and S: The relationships on which the slope effect charts are based were derived from data taken on slopes not exceeding 20% and length not exceeding 400 ft. How far these dimensions can be exceeded before those relationships change has not been determined.

C: More work is needed to improve definitions of cover factor, particularly for areas outside agriculture, such as undistrubed forest, harvested or burned forests, logging roads, mining sites, rangeland, and construction sites.

P: The reported values of the practice factor have been limited to cropland. Definitions of practice factor values are needed for various conservation practices on silviculture, mining, construction and other areas outside agriculture.

S_d: The science of sedimentology has not progressed to the state where the sediment-delivery ratio can be predicted to the degree of accuracy desired.

The loading function in Equation III-1 and supporting data in tables and figures were designed to predict longterm average loadings for specific conditions. Sediment loading for a specific year may be substantially greater or smaller than the annual averages because of differences in number, size, and timing of erosive rainstorms, and in other weather parameters. Table 11 of USDA Agriculture Handbook 282 (1965) contains a listing of 50, 20, and 5% probability values of R factor at 181 key locations in the area east of the Rocky Mountains. These may be used for further characterization of soil-loss hazards.

Due to the uncertainties embedded in factor values, sediment loadings computed by Equation III-1 should be considered as reasonable estimates rather than as absolute data. Table III-9 lists the best estimate of the range of accuracy for Equation III-1 and available supporting data. The range pertains to annual average. For a specific year, the range may be much larger than those given.

TABLE III-9

ESTIMATED RANGE OF ACCURACY OF SEDIMENT LOADS
FROM SURFACE EROSION

Predicted loading (MT/ha/year)	Estimated range of accuracy (MT/ha/year)
0.1	0.001 ~ 1.0
1	0.1 ~ 5
10	5 ~ 15
100	50 ~ 150
1,000	500 ~ 1,500

3.2.4 Source Characteristic Factors for Predicting Maximum and Minimum Sediment Loadings

The loading function in Equation III-1 can be used to predict sediment loading other than annual averages. Variations of the loading rate are embedded in rainfall factor R and cover factor C. The evaluation procedure is illustrated in the following examples.

EXAMPLE III-1

Assessing Sediment Loading from Surface Erosion

The watershed of interest has an area of 830 acres. It is located in Parke County, Indiana. Compute sediment loading from the watershed from sheet and rill erosion in terms of average daily loading, maximum daily loading during a 30 consecutive day period, and minimum during a 30 consecutive day period.

Basic Information

Land use types:
- Cropland
- Pasture
- Woodland

Delivery ratio: 60%

Land information: (Cropland - 180 acres)
- Continuous corn
- Conventional tillage, average yield ∿ 40 to 45 bu
- Cornstalks are left after harvest
- Contour strip-cropped
- Soil - Fayette silt loam
- Slope - 6%
- Slope length - 250 ft

Pasture: (220 acres)
- No appreciable canopy
- Cover at surface - grass and grasslike plants
- Percent of surface or ground cover - 80%
- Soil - Fayette silt loam
- Slope - 6%
- Slope length - 200 ft

Woodland: (430 acres)
- Medium stocked
- Percent of area covered by tree canopy - 50%
- Percent of area covered by litter - 80%
- Undergrowth - managed
- Soil - Bates silt loam
- Slope - 12%
- Slope length - 150 ft

Maximum and Minimum Rates

The ratios between 30-day maximum and average daily rates, and 30-day minimum and average daily rates for continuous corn, pasture, and woodland for this area are evaluated in Examples III-2 and III-3. They are:

Continuous corn: Ratio--30 day maximum/average daily = 3.2
Ratio--30 day minimum/average daily = 0.25

Pasture and woodland: Ratio--30 day maximum/average daily = 2.5
Ratio--30 day minimum/average daily = 0.25

Calculations of Loading Per Acre

Cropland:

$R = 200$ (Figure III-2)

$K = 0.37$ (USDA-SCS)

$LS = 1.08$ (Figure III-8)

$C = 0.49$

$P = 0.25$ (Table III-6)

$S_d = 0.60$

Calculate average annual loading per acre.

$$Y(S)_{annual} = 200 \times 0.37 \times 1.08 \times 0.49 \times 0.25 \times 0.6 = 5.87 \text{ tons/acre/year}$$

Calculate average daily loading per acre.

$$Y(S)_{avg.daily} = 5.87 \text{ tons/acre/year} \div 365 \text{ days} = 0.016 \text{ tons/acre/day} = 32 \text{ lb/acre/day}$$

Calculate maximum loading per acre during a 30 consecutive day period.

$$Y(S)_{30\text{ day max}} = 0.016 \text{ tons/acre/day} \times 3.2$$
$$= 0.052 \text{ tons/acre/day} = 104 \text{ lb/acre/day}$$

Calculate minimum loading per acre during a 30 consecutive day period.

$$Y(S)_{30\text{ day min}} = 0.016 \text{ tons/acre/day} \times 0.25$$
$$= 0.004 \text{ tons/acre/day} = 8 \text{ lb/acre/day}$$

Pasture:

$R = 200$

$K = 0.37$

$LS = 0.95$

$C = 0.013$ (Table III-3)

$P = 1.0$

$S_d = 0.60$

$$Y(S)_{\text{annual}} = 200 \times 0.37 \times 0.95 \times 0.013 \times 1.0 \times 0.6$$
$$= 0.548 \text{ tons/acre/year} = 1{,}100 \text{ lb/acre/year}$$

$$Y(S)_{\text{avg.daily}} = 0.548 \text{ tons/acre/year} \div 365 \text{ days}$$
$$= 0.0015 \text{ tons/acre/day} = 3 \text{ lb/acre/day}$$

$$Y(S)_{30\text{ day max}} = 0.0015 \text{ tons/acre/day} \times 2.5$$
$$= 0.0038 \text{ tons/acre/day} = 7.6 \text{ lb/acre/day}$$

$$Y(S)_{30\text{ day min}} = 0.0015 \text{ tons/acre/day} \times 0.25$$
$$= 0.0004 \text{ tons/acre/day} = 0.8 \text{ lb/acre/day}$$

Woodland:

$R = 200$

$K = 0.32$

$LS = 2.75$

$C = 0.003$ (Table III-5)

$P = 1.0$

$S_d = 0.60$

$$Y(S)_{\text{annual}} = 200 \times 0.32 \times 2.75 \times 0.003 \times 1.0 \times 0.60$$
$$= 0.3168 \text{ tons/acre/year}$$

$$Y(S)_{\text{avg. daily}} = 0.3168 \text{ tons/acre/year} \div 365 \text{ days}$$
$$= 0.0009 \text{ tons/acre/day} = 1.8 \text{ lb/acre/day}$$

$$Y(S)_{\text{30 day max}} = 0.0009 \text{ tons/acre/day} \times 2.5$$
$$= 0.0022 \text{ tons/acre/day} = 4.4 \text{ lb/acre/day}$$

$$Y(S)_{\text{30 day min}} = 0.0009 \text{ tons/acre/day} \times 0.25$$
$$= 0.0002 \text{ tons/acre/day} = 0.4 \text{ lb/acre/day}$$

Calculations of Gross Loading

Average daily:

Cropland - 180 acres x 0.016 tons/acre/day = 2.88 tons/day
Pasture - 220 acres x 0.0015 tons/acre/day = 0.33 tons/day
Woodland - 430 acres x 0.0009 tons/acre/day= 0.39 tons/day

Total $Y(S)_{\text{avg. total}}$ = 3.60 tons/day

30 day maximum:

Cropland - 180 acres x 0.052 tons/acre/day = 9.36 tons/day
Pasture - 220 acres x 0.0038 tons/acre/day = 0.84 tons/day
Woodland - 430 acres x 0.0022 tons/acre/day = 0.95 tons/day

Total $Y(S)_{30 \text{ max total}}$ = 11.15 tons/day

30 day minimum

Cropland - 180 acres x 0.004 tons/acre/day = 0.72 tons/day
Pasture - 220 acres x 0.0004 tons/acre/day = 0.09 tons/day
Woodland -430 acres x 0.0022 tons/acre/day = 0.09 tons/day

Total $Y(S)_{30\text{-min total}}$ = 0.90 tons/day

END OF EXAMPLE III-1

EXAMPLE III-2

Variations Caused by Rainfall Factor Alone

The rainfall erosion index R varies within a year, as shown in percent erosion index curves in Appendix A. For lands where cover factor is relatively constant, such as woodland and grassland, temporal distribution of rainfall factor R governs temporal variations in erosion.

Figure III-11 shows an example of monthly distribution of percentages of annual R values. This distribution curve is for parts of Michigan, Missouri, Illinois, Indiana, and Ohio based on Curve 16 in Figure A-2d. The following steps are required for evaluating monthly variation of R values.

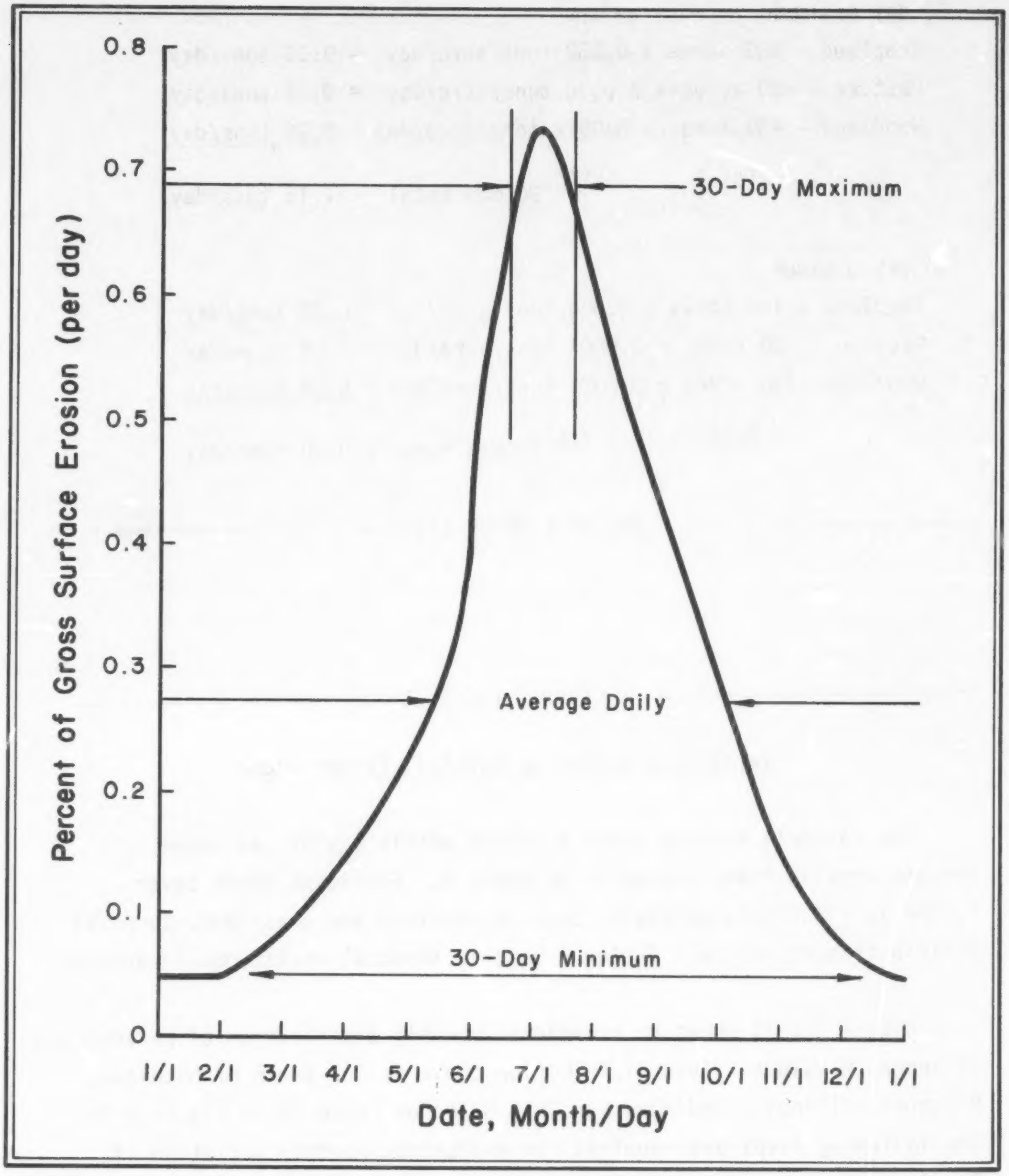


FIGURE III-11 PROJECTED VARIATION OF SOIL EROSION FOR LANDS WITH CONSTANT COVER FACTOR, IN PARTS OF MICHIGAN, MISSOURI, ILLINOIS, INDIANA, AND OHIO (MCELROY, ET AL., 1976)

1. Read the percent of annual erosion index, at the predetermined time interval, on the appropriate erosion index distribution curve (for this specific example, Curve 16 on Figure A-2d in Appendix A).

2. For each time interval, subtract the reading of the first date from that of the last date.

3. Results of Step 2 are the percents of annual index that are to be expected within the particular periods. Use these data for plotting distribution curve. The percent average daily is 0.274, which is obtained by dividing 100 (percent) by 365 (days in a year).

The curve in Figure III-11 indicates that, if other factors hold constant, soil erosion in this area would have its maximum from 20 June to 20 July, and minimum from late December to late January.

One estimates that, based on the R distribution in Figure III-11, the maximum daily loading rate during a 30-consecutive-day period for woodland and grassland in this particular area is approximately 2.5 times that of average daily loading rate for 1 year; the minimum daily rate during a 30-consecutive-day period is approximately one-fourth of the average daily rate.

END OF EXAMPLE III-2

EXAMPLE III-3

Variations Caused by the Combined Effects of Rainfall Factor and Cover Factor

For croplands, where soils are tilled and surface conditions change drastically from one crop stage to another, evaluation of erosion variation should include both the R factor and C factor.

Required steps to achieve such evaluations are:

1. Determine average dates of each crop stages.
2. Determine C factor values for each crop stage from such information as productivity, disposition of crop residues and tillage.
3. Obtain monthly distribution of R.
4. Multiply C factor values by the R value of the corresponding period.

Variations of RC products are the temporal variations of sediment loading.

In this example, temporal variation of surface erosion rate for continuous cornland in central Indiana was calculated. Again, the erosion index distribution Curve No. 16 on Figure A-2d was used. Assumptions were conventional tillage, a yield average of 40 to 59 bu of corn per acre, and cornstalks left on the field after harvesting. The dates, C values, and percent of erosion index for five-crop stages, and RC products are:

		Percent R		
Crop stage, starting-ending date	Cover factor, C[a/]	Reading[b/]	Percent in the Period	RC product
Turn plowing, 5/1-5/19	0.55	13.8	5.7	3.14
Seeding, 5/20-6/19	0.70	19.5	16.5	11.55
Establishment, 6/20-7/19	0.58	36.0	21.3	12.35
Growing crop, 7/20-10/9	0.32	57.3	33.7	10.78
Harvest and stubble, 10/10-4/30	0.50	91.0	22.8	11.40
Total			100	49.22

a/ Reference source: USDA-Agricultural Research Service Handbook No. 282[9/] Table 2.

b/ Reading from Figure A-2d (Curve 16) for starting date.

The annual C factor is estimated at 0.49. Temporal variation of surface erosion rate, in terms of percent of annual total, is shown in Figure III-12. It is seen that the maximum erosion from this continuous cornland would occur in mid-June through mid-July, nearly identical to the period of maximum erosion with constant soil cover (Figure III-11). The 30-day maximum is approximately 3.2 times average daily, which is higher than the previous (constant C factor) estimate due to the magnifying effect caused by the overlapping of a high R period with a high C period. Figure III-12 also shows that minimum erosion would occur during the winter season; the 30-day minimum is one-fourth of the average daily load.

END OF EXAMPLE III-3

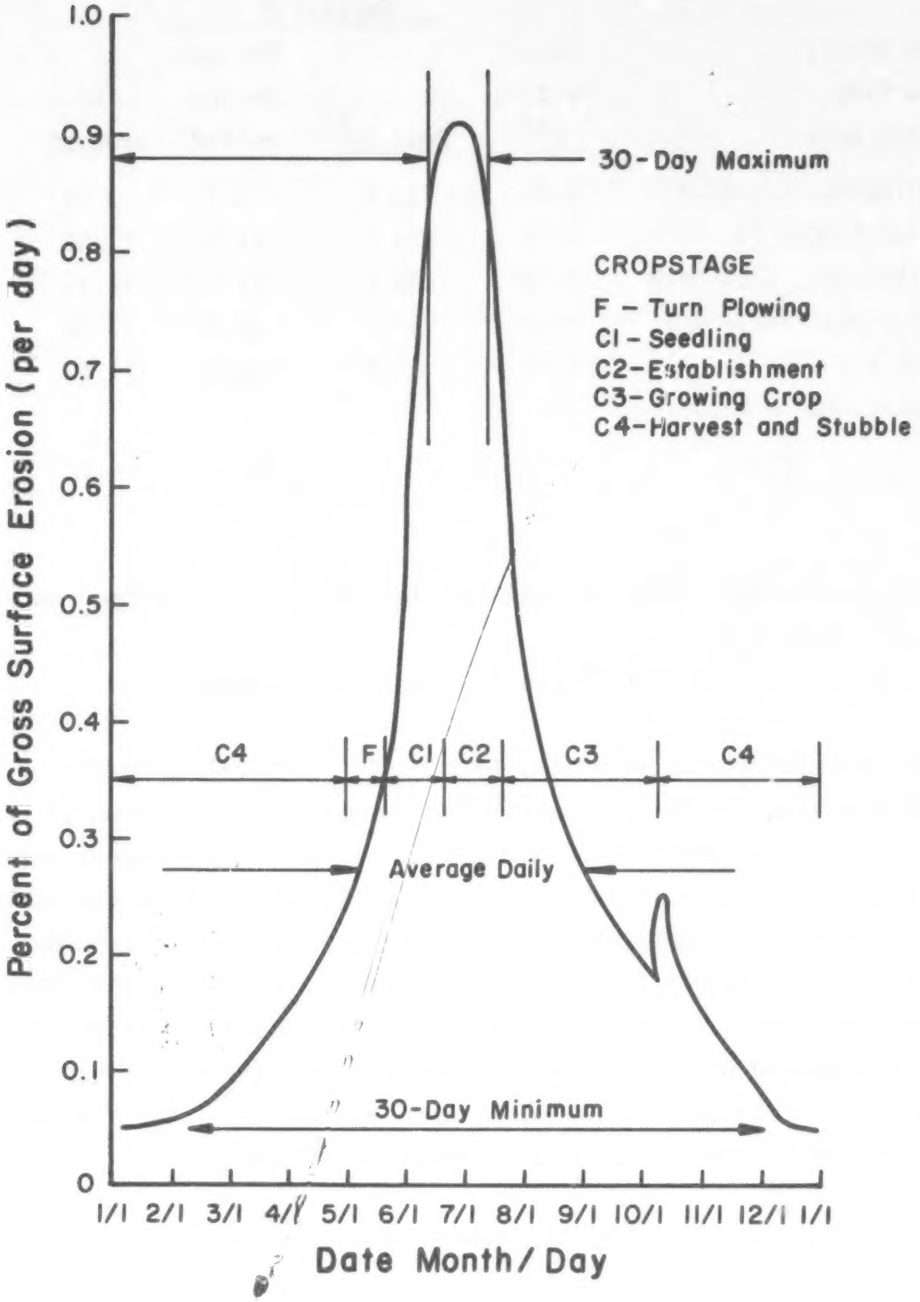


FIGURE III-12 PROJECTED VARIATION OF SOIL EROSION ON CONTINUOUS CORN LANDS IN CENTRAL INDIANA (MCELROY ET AL., 1976

3.3 NITROGEN LOADING FUNCTION

While the complex interactions in soil, air, water and plants are reasonably well understood, methods for quantifying movements within the system are still in the research stage. Methods which are suitable for general use often oversimplify the problem. They must be used with discretion and may be quite inadequate in certain cases. In particular, it is not presently possible to describe leaching processes for soluble forms of nitrogen. The nitrogen loading function is made up of two sources: (a) erosion; and (b) precipitation. Total nitrogen loading is obtained by adding the yields from both sources. The loading functions exclude leaching losses, and predict the amount of total nitrogen that is released to surface waters by runoff and erosion. The nitrogen in precipitation is mostly in available form.

3.3.1 Nitrogen Loading Function for Erosion Loss

The nitrogen loading from erosion is computed as:

$$Y(NT)_E = a \cdot Y(S)_E \cdot C_S(NT) \cdot r_N \qquad \text{(III-3)}$$

where

$Y(NT)_E$ = total nitrogen loading from erosion, kg/year (lb/year)

a = dimensional constant (10 for metric units, 20 for English units)

$C_S(NT)$ = total nitrogen concentration in soil, g/100 g

$Y(S)_E$ = sediment loading from surface erosion, MT/year (tons/year)

r_N = nitrogen enrichment ratio

Available nitrogen can be obtained by using a fraction f_N which is the ratio of available N to total N in sediment. Thus, the available nitrogen in sediment is

$$Y(NA)_E = Y(NT)_E \cdot f_N \qquad \text{(III-4)}$$

3.3.2 Nitrogen Loading Function for Precipitation

The nitrogen loading from precipitation is computed as:

$$Y(N)_{Pr} = A\cdot\frac{Q(OR)}{Q(Pr)}\cdot N_{Pr}\cdot b \qquad \text{(III-5)}$$

where

$Y(N)_{Pr}$ = stream nitrogen load from precipitation, kg/year (lb/year)

A = area, ha (acres)

Q(OR) = overland flow from precipitation, cm/year (in/year)

Q(Pr) = total amount of precipitation, cm/year (in/year)

N_{Pr} = nitrogen load in precipitation, kg/ha/year (lb/acre/year)

b = attenuation factor (use b = 0.75 for metric and English units unless better information is available).

Almost all of $Y(N)_{Pr}$ will be in the available form so that the total available nitrogen from both erosion and precipitation may be obtained by adding Equations III-4 and III-5. Thus,

$$Y(NA) = Y(NT)_E\cdot f_N + Y(N)_{Pr} \qquad \text{(III-6)}$$

3.3.3 Evaluation of Parameters in the Nitrogen Loading Function

The value of $Y(S)_E$ can be evaluated from the sediment loading function presented in Section 3.2. The value of the enrichment ratio r_N is variable according to the soil texture and cultural treatment. Viets (1971) presented the values of r_N using data from small experimental plots (see Table III-10). Hagin and Amberger (1974), as well as Stoltenberg and White (1955) have proposed an r_N value of 2.0. Massey, et al., (1953) estimated the value of r_N as 2.7. Because of wide variations in the properties of erodible soil, a single value of r_N is not probable; the values reported range from less than 2.0 to greater than 4.0 and an appropriate value should be selected for a specific location from local knowledgeable sources such as the State Agricultural Experiment Stations.

TABLE III-10

NUTRIENT AND SEDIMENT LOSSES (VIETS, 1971)

Source	Total loss (kg/ha) Soil	N	P	Enrichment ratio, r N	P
Check	29,100	74.5	75.8	3.88	1.59
Rye winter cover crop	13,160	38.9	37.7	4.08	1.56
Manure (45 MT/ha)	18,390	52.8	44.3	4.28	1.47
Rye and manure (45 MT/ha)	8,130	21.5	19.6	3.35	1.47

The value of $C_S(NT)$ in the plowed layer of soil is variable from location to location and from time to time. Estimates of native soil nitrogen in the U.S. indicate a range between 0.02 and 0.4% (Jenny, 1930). Parker et al.* (1946) published a map in 1946 showing the nitrogen content in the top 1-ft layer in the U.S. (see Figure III-13). Data in Figure III-13 should be viewed in general terms; for specific sites, local sources such as SAES and SCS Soil Survey should be consulted.

Precipitation also contributes to the soil nitrogen. Atmospheric nitrogen extracted by soil microbes becomes incorporated into soil organic matter; animal manures, crop residues, and other wastes contribute significant amounts of nitrogen to the soil. Jenny (1930) expressed the nitrogen content of the soil in terms of temperature, T, and a humidity factor, H. Jenny's equation is:

$$C_S(NT) = 0.55_e{}^{-0.08T} (1 - e^{-0.005H}) \quad \text{(III-7)}$$

$$H = \frac{P}{(1 - \frac{RH}{100}) SVP_t} \quad \text{(III-8)}$$

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* Note that in areas of heavy antecedent cultivation, particularly where large amounts of anhydrous ammonia or other nitrogen sources are applied, these estimates may not be valid.

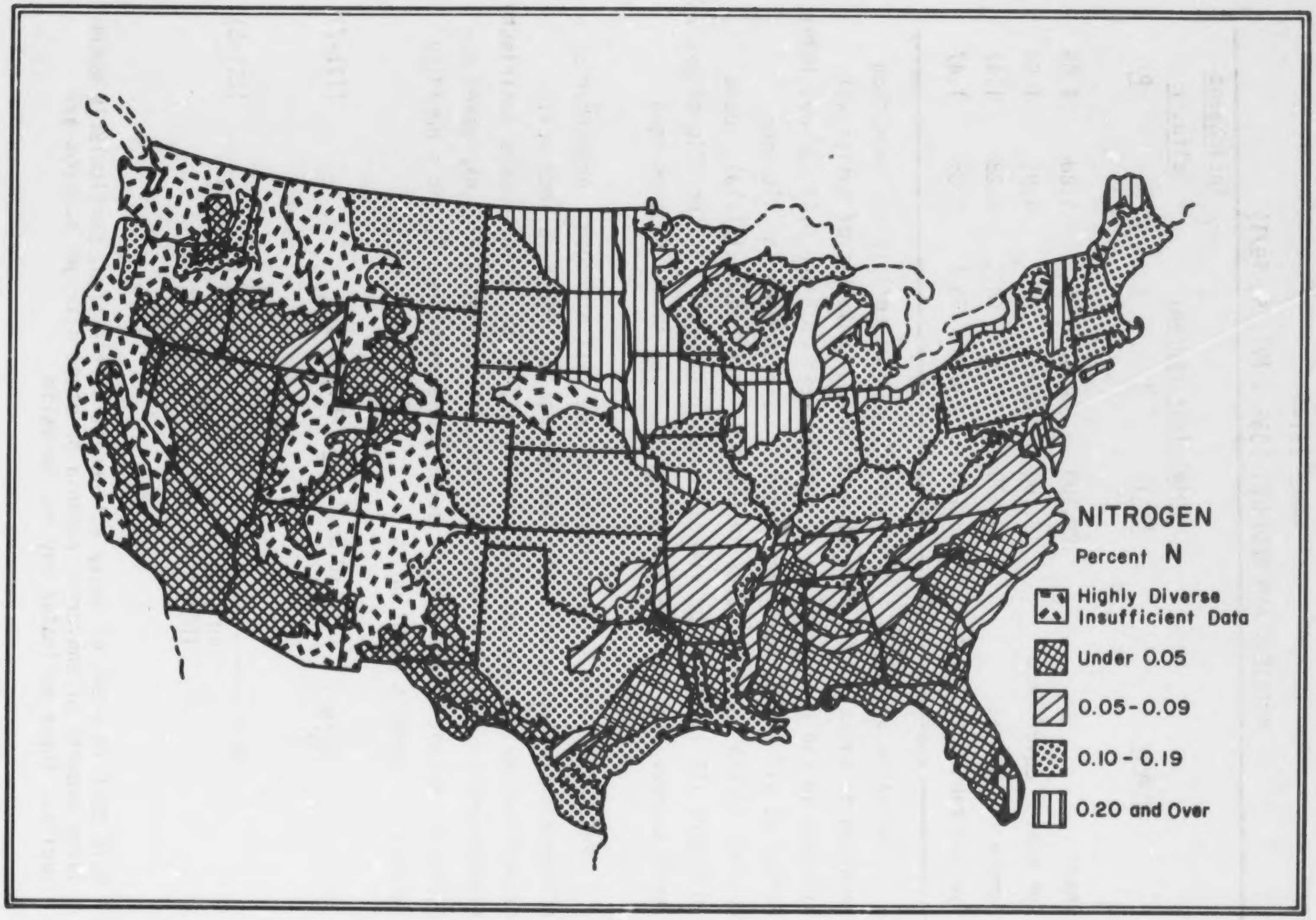
NITROGEN
Percent N
Highly Diverse Insufficient Data
Under 0.05
0.05-0.09
0.10 - 0.19
0.20 and Over

FIGURE III-13 PERCENTAGE NITROGEN (N) IN SURFACE FOOT OF SOIL (PARKER, ET AT., 1946)

where

P = precipitation, mm/year

$C_S(NT)$ = concentration of soil nitrogen, g/100 g

T = annual average temperature, ^oC

RH = relative humidity, %

SVP_t = saturated vapor pressure at given temperature, mm of Hg

Equation III-9 shows the relation between SVP_t and T.*

$$SVP_t = 10^{\left[9.2992 - 2360/(273 + T)\right]} \qquad (III\text{-}9)$$

The solution of Equation III-7 is shown graphically in Figure III-14. The value of humidity factor, H, can be determined from Equations III-8 and III-9. A nomograph solution of H is shown in Figure III-15. For given values of precipitation, relative humidity and temperature, the value of H can be quickly and accurately established from Figure III-15. For example, given P_1 = 500 mm/year (19.7 in/year), RH_1 = 60%, and $T_1 = 5^oC$ (41^oF), the value of H factor can be determined as follows: using a straight-edge ruler, align P_1 and RH_1 to intersect on the index line at "a" as shown on the inset of Figure III-15. Align "a" with T_1 on the temperature scale to intersect the H scale. The result on the H scale is 194.

Data in Figure III-13 may be used as a check on current data. Equations (III-8) and (III-9) may be used to calculate nitrogen content of soil more precisely if necessary data are available for using these equations. Again data from State Agricultural Experiment Stations, and SCS Soil Surveys are much more dependable than the above sources and should be consulted whenever possible.

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* Modified from Gladstone, S., Elements of Physical Chemistry, D. Van Nostrand Company, Inc., New York, New York (1946)

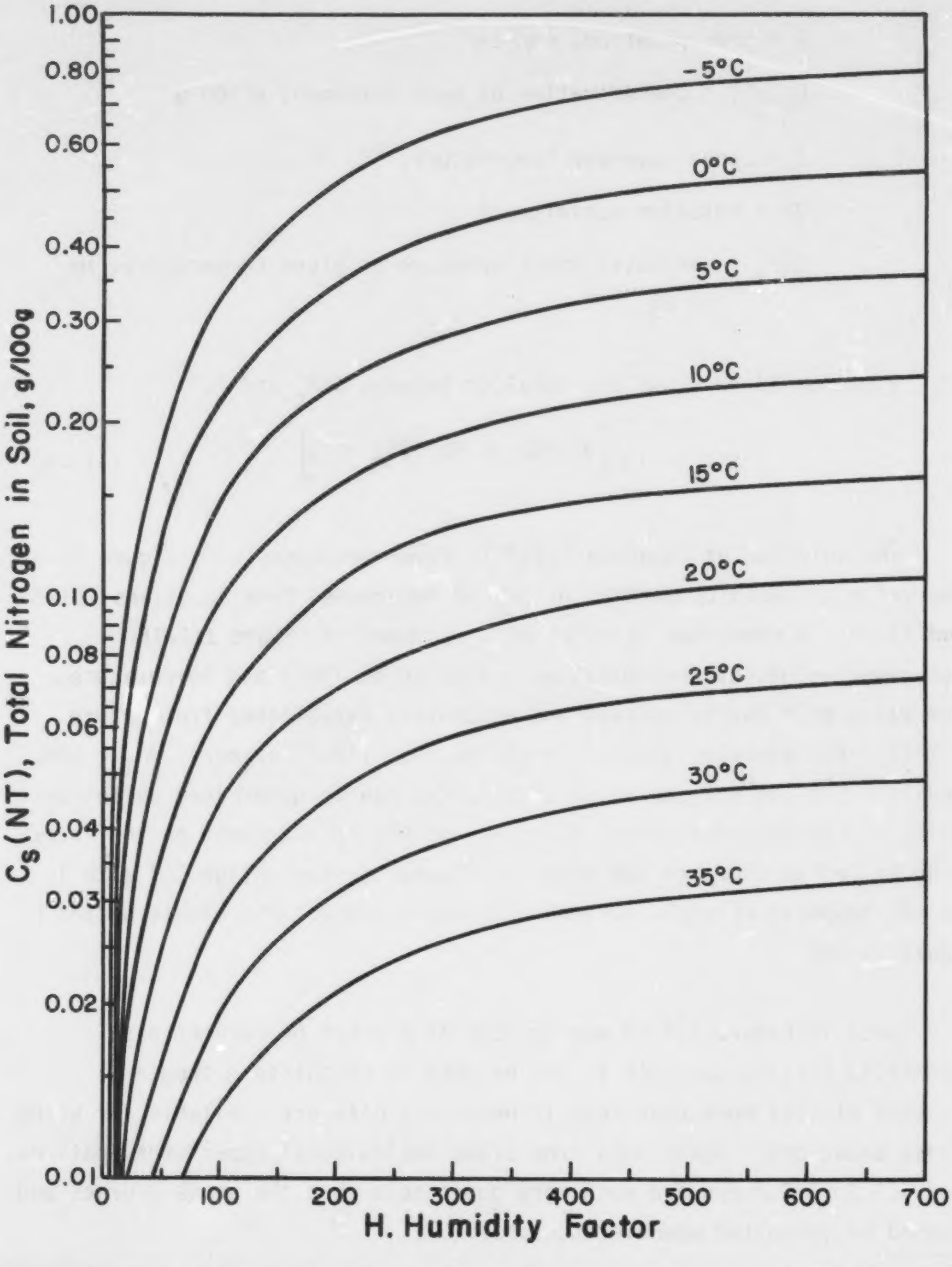
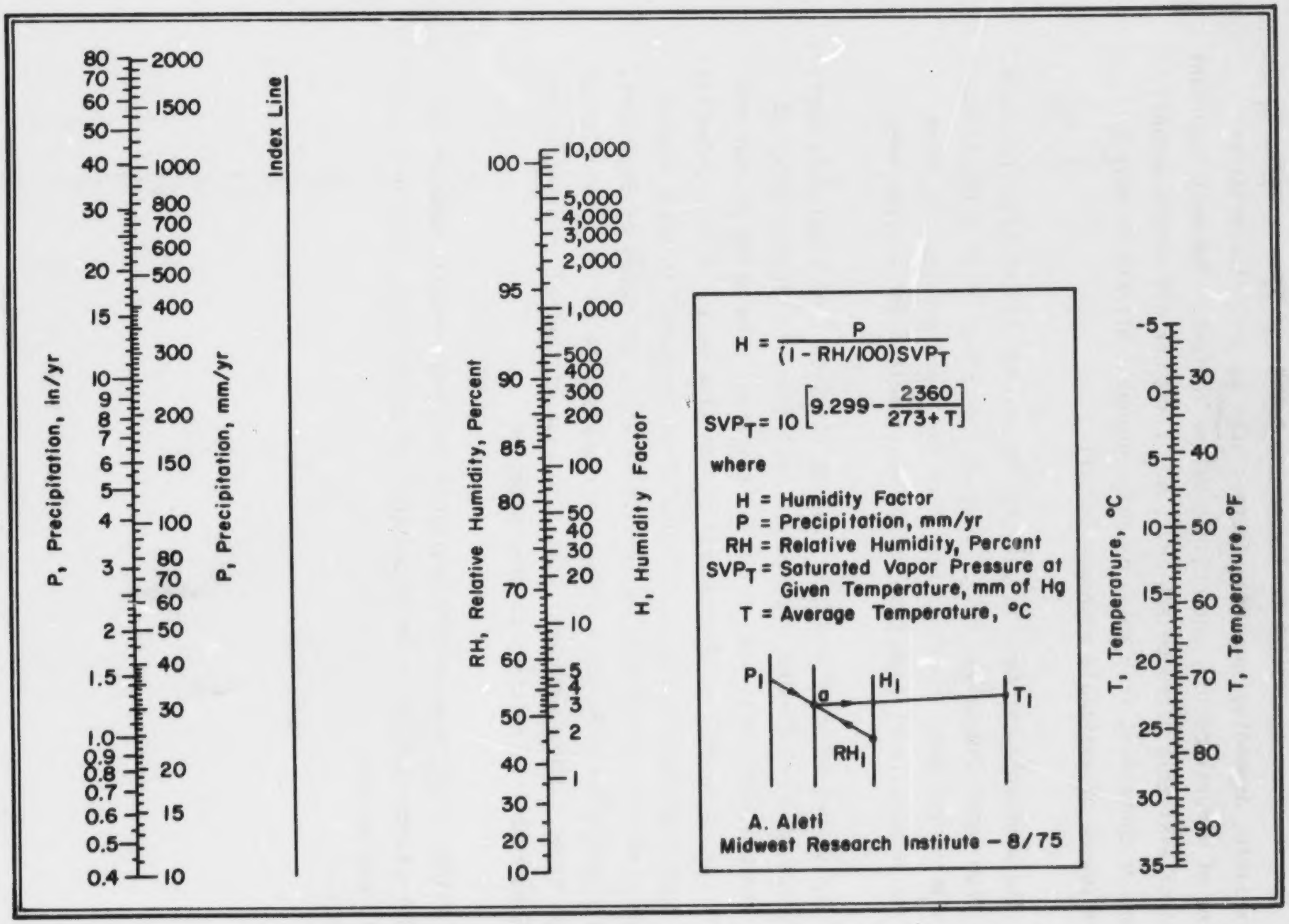


FIGURE III-14 SOIL NITROGEN VS. HUMIDITY FACTOR AND TEMPERATURE



$$H = \frac{P}{(1 - RH/100)SVP_T}$$

$$SVP_T = 10^{\left[9.299 - \frac{2360}{273+T}\right]}$$

where

H = Humidity Factor
P = Precipitation, mm/yr
RH = Relative Humidity, Percent
SVP_T = Saturated Vapor Pressure at Given Temperature, mm of Hg
T = Average Temperature, °C

A. Aleti
Midwest Research Institute – 8/75

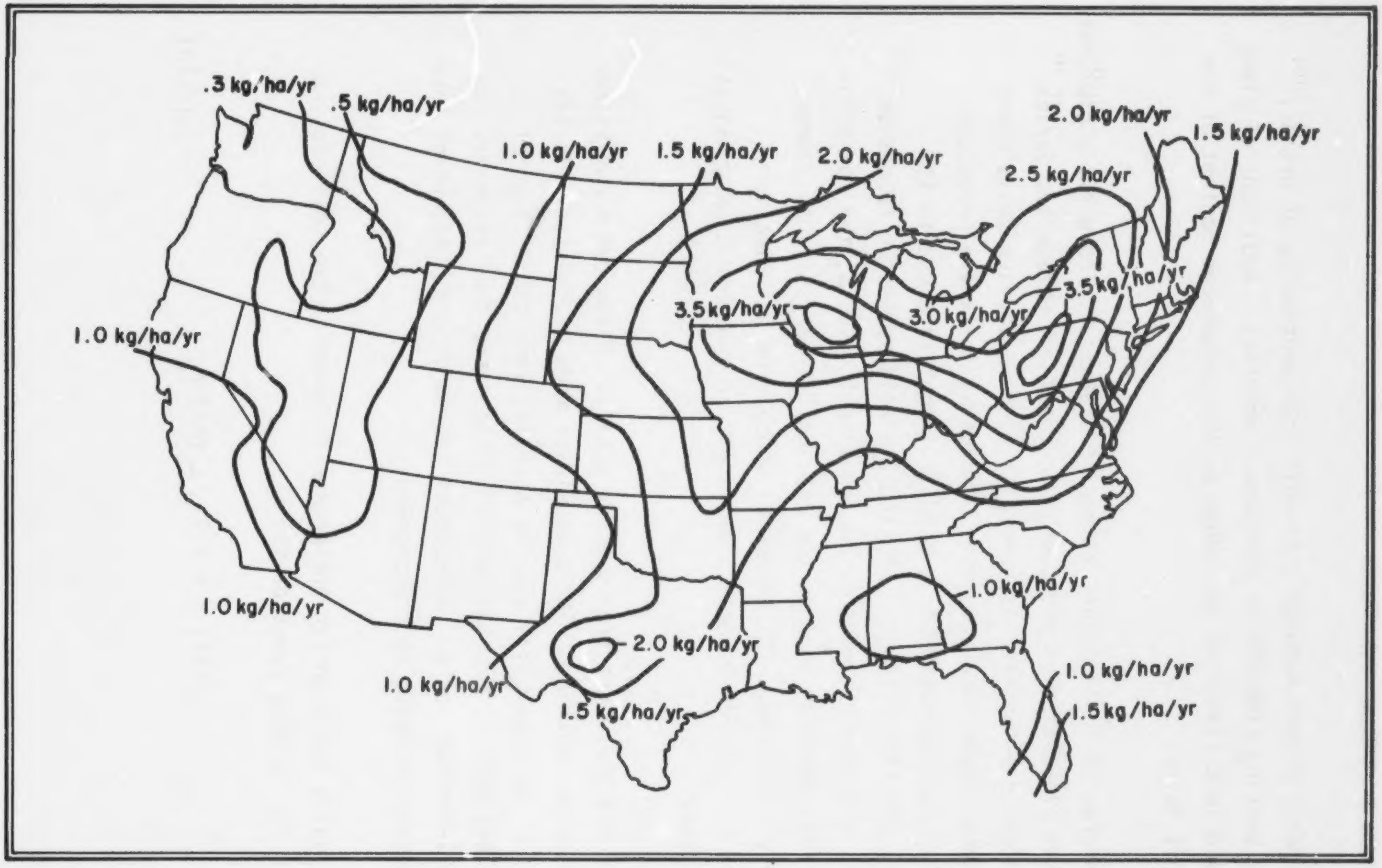
FIGURE III-15 NOMOGRAPH FOR HUMIDITY FACTOR, H

The fraction of available nitrogen to total nitrogen in soil, f_N is variable, depending upon many factors such as soil chacteristics, degree of mineralization, and organic matter content. The most important forms of available nitrogen are NH_4^+, NO_3^-, and certain simple organic compounds containing free amide or amino groups. Nitrate is only a minor source of available nitrogen in soil.

The available nitrogen in soil rarely exceeds 15% of total nitrogen. Data from Lopez and Galvez (1958) suggest that about 8% of total nitrogen in soil is available in mineralized form for plant growth. For more precise values, local expertise should be consulted for a given area.

The values of Q(OR) and Q(Pr) may be obtained from local data sources. The value of Q(Pr) (annual average precipitation) is usually obtained from the weather bureau statistics for the area. The value of overland runoff can be roughly estimated from stream flow data. A user unfamiliar with hydrology should consult with qualified personnel in state conservation services, agricultural extension service, the Corps of Engineers, or the Agricultural Research Service for assistance in interpretation of stream flows. These resources will also have historical information on overland runoff in relation to precipitation.

Values of N_{Pr} are usually available from measurements made in the local research stations. In the absence of actual data, data in Figure III-16 may be used.


.3 kg/ha/yr
.5 kg/ha/yr
1.0 kg/ha/yr
1.5 kg/ha/yr
2.0 kg/ha/yr
2.0 kg/ha/yr
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1.0 kg/ha/yr
2.0 kg/ha/yr
1.0 kg/ha/yr
1.5 kg/ha/yr
1.0 kg/ha/yr
1.5 kg/ha/yr

FIGURE III-16 NITROGEN (NH_4-N AND NO_3-N) IN PRECIPITATION. (PERSONAL COMMUNICATION WITH MRI, J.H. CRAVENS, REGIONAL RORESTER, U.S.D.A.-FS, EASTERN REGION, 1974)

3.4 PHOSPHORUS LOADING FUNCTION

Phosphorus occurs naturally in soil from weathering of primary phosphorus-bearing minerals in the parent material. Additions of plant residues and fertilizers by man enhances the phosphorus content of the surface soil layer.

Phosphorus in soils occurs either as organic or inorganic phosphorus. The relative proportion of phosphorus in these two categories varies widely. Organic phosphorus is generally high in surface soils where organic matter tends to accumulate. Inorganic forms are prevalant in subsoils. Soil phosphorus is readily immobilized due to its affinity to certain minerals. In strongly acid soils the formation of iron and aluminum phosphates, and in alkaline soils, the formation of tricalcium phosphate reduces the availability of soil phosphorus. Once it enters a stream, the nature of phosphorus existing in sediment or in solution becomes significant in the nutrition of aquatic microorganisms.

Phosphorus transport from a given site to stream can occur either by erosion or by leaching. The predominant mode of transport is via soil erosion. Soil solution usually contains less than 0.1 μg of phosphorus per milliliter; the leaching losses are thus extremely low even in well-drained soils. Exceptions are sands and peats which have little tendency to react with phosphorus.

The loading function for phosphorus is based on the soil erosion mechanism. The loading function is:

$$Y(PT) = a \cdot Y(S)_E \cdot C_S(PT) \cdot r_P \qquad \text{(III-10)}$$

where

$Y(PT)$ = total phosphorus loading, kg/year (lb/year)

a = a dimensional constant (10 metric, 20 English)

$Y(S)_E$ = sediment loading, MT/year (tons/year)

$C_S(PT)$ = total phosphorus concentration in soil, g/100 g

r_p = phosphorus enrichment ratio

Available phosphorus may be computed from Equation III-11:

$$Y(PA) = Y(PT) \cdot f_p \tag{III-11}$$

where

$Y(PA)$ = yield of available phosphorus, kg/year (lb/year)

f_p = ratio of available phosphorus to total phosphorus

3.4.1 Evaluation of Parameters in Phosphorus Loading Function

Sediment loading, $Y(S)_E$, may be obtained from procedures outlined in Section 3.2.

The value of $C_S(PT)$, the total phosphorus content of the soil, is variable. For any given location, current and local data are preferred to generalized values given in this report. No central repository of current nationwide data exists. Parker, et al., (1946) published data on the phosphorus content of soil in the top 30 cm (1 ft) for the 48 states, as shown in Figure III-17. Parker's data, although obtained 30 years ago, will serve as a check on current data. Soil surveys periodically made by the Soil Conservation Service contain more recent information on soil phosphorus content. State agricultural extension service personnel can also provide reasonable estimates of soil phosphorus content in a given area. These sources should be given priority in

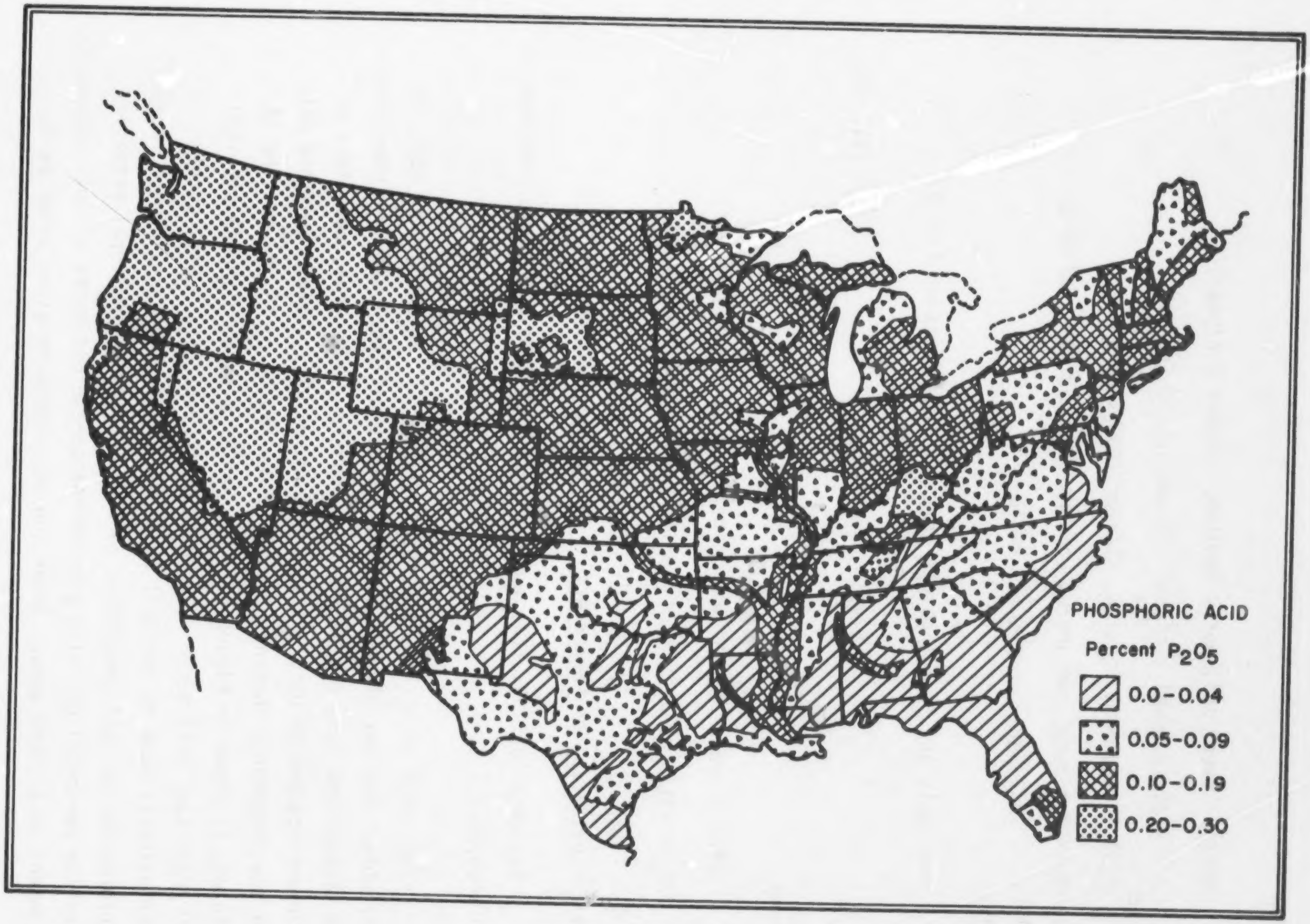
PHOSPHORIC ACID
Percent P_2O_5
0.0-0.04
0.05-0.09
0.10-0.19
0.20-0.30

FIGURE III-17 PHOSPHORUS CONTENT IN THE TOP 1 FT OF SOIL (PARKER, ET AL., 1946)

determining the phosphorus content of the soil.

The enrichment ratio, r_p, has been the least researched parameter in the loading function. As shown in Table III-10, the reported r_p values average about 1.5. Massey, et al., (1953) obtained an r_p value of 3.4, and Stoltenberg and White (1955) reported a value of 2.0. Hagin and Amberger (1974) have used a value of r_p of 2.5 in their simulation model for nutrient losses from agricultural sources. Massey, et. al., (1955) have developed an empirical equation to determine r_p:

$$\log r_p = 0.319 + 0.25\,(-\log X) + 0.098\,(-\log Y) \qquad \text{(III-12)}$$

where

X = sediment loss/unit of runoff, tons/acre-in
Y = sediment loss, tons/acre

The determination of available phosphorus in the soil is difficult. Most reported data fail to distinguish between soluble phosphorus, absorbed or particulate phosphorus and organic phosphorus in sediment runoff. Total phosphorus is not always a useful parameter, since only the soluble orthophosphate form is immediately available for uptake by aquatic organisms. However, other forms of phosphorus in sediment can act as a source or sink for subsequent release in available form.

Schuman, et. al., (1970) have reported an empirical relation between sediment phosphorus (concentration in ppm, $C_S(PT)$) and soluble phosphorus (concentration in ppm, $C_Q(P)$) for Iowa soils. The relation may be stated as:

$$C_Q(P) = a + b \cdot C_S(PT) \qquad \text{(III-13)}$$

where a and b are regression coefficients. The reported values of a and b are 0.018 and 0.047, respectively. Equation III-13 shows that the ratio of solution phosphorus to sediment phosphorus is just under 1:20.

Taylor (1967) suggested that about 10% of the total phosphorus in eroded soil is ordinarily available for aquatic plant growth.

3.5 ORGANIC MATTER LOADING FUNCTION

The loading function is:

$$Y(OM)_E = a \cdot C_S(OM) \cdot Y(S)_E \cdot r_{OM} \qquad \text{(III-14)}$$

where

$Y(OM)_E$ = organic loading, kg/year (lb/year)

a = a dimensional constant (10 metric, 20 English)

$C_S(OM)$ = organic matter concentration of soil, g/100 g

$Y(S)_E$ = sediment loading, MT/year (tons/year)

r_{OM} = enrichment ratio for organic matter in eroded soil

3.5.1 Evaluation of Parameters in the Organic Matter Loading Function

The value of $Y(S)_E$ can be obtained from procedures discussed previously. The value of $C_S(OM)$ should be obtained preferably from current or historical data for a given area, (e.g. from the extension service). For approximate values, $C_S(OM)$ may be taken as equal to 20 x $C_S(NT)$, where $C_S(NT)$ is the total nitrogen concentration in the soil. (Buckman and Brady, 1969).

The value of r_{OM}, the enrichment ratio, is more difficult to assess due to lack of research data. Values of r_{OM} are in the range of 1 to 5. The enrichment ratio for sandy soils will be high. Conversely, the enrichment ratio will be low when the mineral fraction of the soil is finely divided and highly erodible. The user should consult with local soil experts and should use local data when available.

EXAMPLE III-4

Example of Loading Computation for Nitrogen, Phosphorus, Organic Matter

The watershed used in Example III-1 (Section 3.2.4) for Parke County, Indiana will be used to illustrate the methodology presented in this section for computing the pollutant loads. It is required to compute available nitrogen, available phosphorus, and organic matter loading for the given area for the following conditions:

Average daily loading;
Maximum daily loading during a 30 consecutive day period; and
Minimum daily loading during a 30 consecutive day period.

The following data, plus soil data, are required:

Soil nitrogen content.
Soil phosphorus content.

The preferred source of data is local records. Jenny's equation (Eq. III-7) and Figure III-17 are alternate sources from which general values may be estimated.

Nitrogen Loading

Using the following data, soil nitrogen content is calculated;

Average annual temperature = $10^{o}C$
Average annual precipitation = 96.5 cm
Average annual relative humidity = 70%

Using the nomograph given in Figure III-15, the value of H factor was determined to be 350. From Figure III-14, and using H = 350 and $T = 10^{o}C$, the value of $C_S(NT)$, the soil nitrogen content was estimated to be 0.204% or 0.204 g/100 g. Assume that 6% of total nitrogen is available, and r_N is 2.0. Using Equations III-3 and III-4

$$\begin{aligned} Y(NA)_E &= 20 \cdot Y(S)_E \cdot 0.204 \cdot 2.0 \cdot 0.06 \\ &= 0.49 \cdot Y(S)_E \end{aligned} \qquad \text{(III-15)}$$

The values of areal sediment yield as given in Example III-1 are shown below in Table III-11.

TABLE III-11
SEDIMENT YIELD IN EXAMPLE

	Sediment yield (tons/day)		
Land Use	Daily Average	Maximum 30 days	Minimum 30 days
Cropland	2.88	9.36	0.72
Pasture	0.33	0.84	0.09
Woodland	0.39	0.95	0.09
Total	3.60	11.15	0.90

The nitrogen loadings are shown in Table III-12 using the data in Table III-11 and Equation III-15.

TABLE III-12
AVAILABLE NITROGEN LOADING (LB/DAY)

		Nitrogen Loading (lb/day)	
Land Use	Daily Average	Maximum 30 days	Minimum 30 days
Cropland	1.41	4.59	0.35
Pasture	0.16	0.41	0.04
Woodland	0.19	0.47	0.04
Total	1.76	5.47	0.43

Phosphorus Loading

Assume $C_S(PT) = 0.255$ g/100 g for the area, 10% of $C_S(PT)$ is available phosphorus, $c_S(PA)$; and r_P is 1.5, and using Equation III-10;

$$Y(PA)_E = 20 \cdot Y(S)_E \cdot 0.255 \cdot 1.5 \cdot 0.10 \qquad \text{(III-16)}$$
$$= 0.765\ Y(S)_E$$

Phosphorus loadings computed from Table III-9 and Equation III-10 are shown in Table III-13.

TABLE III-13
AVAILABLE PHOSPHORUS LOADING (LB/DAY)

Land Use	Daily Average	Maximum 30 days	Minimum 30 days
Cropland	2.20	7.16	0.55
Pasture	0.25	0.64	0.07
Woodland	0.30	0.73	0.07
Total	2.75	8.53	0.69

Organic Matter Loading

Using Equation III-14, data for $C_S(OM)$, $Y(S)_E$, r_{OM} are needed.

Assume that the value of $C_S(OM)/C_S(NT)$ equals 20 and $r_{OM} = 2.5$,

$$Y(OM)_E = 20 \cdot 2.5 \cdot Y(S)_E \cdot 20 \cdot C_S(NT)$$
$$= 1000 \cdot C_S(NT) \cdot Y(S)_E$$

Using $C_S(NT) = 0.2\%$,

$$Y(OM)_E = 200 \cdot Y(S)_E \qquad \text{(III-17)}$$

The values of organic loading are computed from Equation III-17 and presented in Table III-14.

TABLE III-14

ORGANIC MATTER LOADINGS (LB/DAY)

Land Use	Daily Average	Maximum 30 days	Minimum 30 days
Cropland	576	1,872	144
Pasture	66	168	18
Woodland	78	190	18
Total	720	2,230	180

END OF EXAMPLE III-4

3.6 ACCURACY OF NUTRIENT AND ORGANIC MATTER LOADING FUNCTIONS

The accuracy of predicting loads using the loading functions presented in the preceding sections depends, to a large extent, on the availability of reasonably accurate data for evaluating the various parameters in the functions. For example, the nitrogen loading function is composed of several parameters, each of which is in turn a function of several other variables. In addition, several options are available to the user to develop the parameter values from his own sources of information which may alter the prediction accuracy. However, if the used values reflect the longterm average rather than a specific year, and if reasonably large areas are used such as large watersheds (> 100 square miles) rather than individual plots or small watersheds, the expected accuracy can be reasonably estimated. Using the logic that the error in individual parameters will tend to accumulate to a larger error, the expected ranges of predicted values for given "true" or estimated values of load are presented in Table III-15.

These estimates are more accurate for nitrogen when erosion is moderate to extensive, and are less accurate when erosion is slight or when surface runoff is negligible. In the latter cases dissolved

TABLE III-15

PROBABLE RANGE OF LOADING VALUES FOR NUTRIENTS AND ORGANIC MATTER

Loading Function	Estimated value (kg/ha/year)	Probable range (kg/ha/year)
Total N sediment[a]	1	0.1-10
Total N sediment	10	5-20
Total N sediment	50	30-75
Total N precipitation[b]	0.3	0.1-0.6
Total P[c]	1.0	0.5-3.0
Total P	5.0	2-10
Total P	10.0	5-20
Organic matter	10.0	5-20
Organic matter	100	50-200

[a] Available N in sediment will range from 3 to 8% of total N.
[b] Available N is equal to total N in precipitation.
[c] Available P in sediment will range from 5 to 10% of total P.

forms of nitrogen are the principal nitrogen pollutants. These forms are transported either to subsurface waters or directly to surface waters in runoff. Functions which describe either of the latter phenomena are not yet available, and the approach to estimating dissolved forms of nitrogen accordingly involves a combination of local or regional experience supplemented by measurements of soluble nitrogen forms in runoff and baseflow.

Nutrient and organic matter loading functions presented in this section are accordingly based on the sediment loading function developed previously. It is assumed that the nutrients and organic matter are carried through surface runoff and that most of these materials are removed with sediment.

Because the currently available data applicable to the entire U.S. may not reflect the local conditions, it is suggested that local data whenever available be used in preference to the general data presented in this section.

3.7 LOADING VALUES FOR SALINITY LOADS IN IRRIGATION RETURN FLOW

Perhaps the most useful method of establishing salinity loads is through loading values determined for particular regions. Lists of such values are presented in Tables III-16 through III-20 for subbasins in the Colorado River basin, and for irrigated regions in California.

Studies in the Twin Falls area and the Colorado River basin indicate that the range of values for salt pickup from irrigated lands is roughly 1.3 to 22 MT/ha/year (0.5 to 8 tons/acre/year) (Skogerboe and Law, 1971). An average salt pickup rate might be 5 MT/ha/year (2 tons/acre/year). On a per day basis, the range becomes 3 to 50 kg/ha/day (3 to 44 lb/acre/day), and the average becomes 12 kg/ha/day (11 lb/acre/day).

TABLE III-16

SALT YIELDS FROM IRRIGATION IN GREEN RIVER SUBBASIN (EPA, 1971)

Area	Average salt yield		
	(tons/acre/yr)	(kg/ha/day)	(lb/acre/day)
Green River above New Fork River	0.1	0.6	0.5
Big Sandy Creek	5.6	34.3	30.7
Blacks Fork in Lyman area	2.4	14.7	13.2
Hams Fork	0.3	1.8	1.6
Henry's Fork	4.9	30.1	26.9
Yampa River above Steamboat Springs	0.2	1.2	1.1
Yampa River, Steamboat Springs to Craig	0.4	2.5	2.2
Mill Creek	0.1	6.1	5.4
Williams Fork River	0.3	1.8	1.6
Little Sanke above Dixon	0.3	1.8	1.6
Little Sanke, Dixon to Baggs	0.5	3.1	2.7
Ashley Creek	4.2	25.8	23.0
Duchesne River	3.0	18.4	16.4
White River below Meeker	2.0	12.3	11.0
Price River	8.5	52.2	46.6
San Rafael River	2.9	17.8	15.9

TABLE III-17

SALT YIELDS FROM IRRIGATION IN UPPER COLORADO MAIN STREAM SUBBASIN (EPA, 1971)

Area	Average salt yield (tons/acre/yr)	Average salt yield (kg/ha/day)	Average salt yield (lb/acre/day)
Main stem above Hot Sulphur Springs	0.3	1.8	1.6
Main stem, Hot Sulphur Springs to Kremmling	0.9	5.5	4.9
Muddy Creek Drainage Area	2.4	14.7	13.2
Brush Creek	0.7	4.3	3.8
Roaring Fork River	3.5	21.5	19.2
Colorado River Valley, Glenwood Springs to Silt	2.3	14.1	12.6
Colorado River, Silt to Cameo	3.5	21.5	19.2
Grand Valley	8.0	49.1	43.8
Plateau Creek	0.9	5.5	4.9
Gunnison River above Gunnison	0.3	1.8	1.6
Tomichi Creek above Parlin	0.3	1.8	1.6
Tomichi Creek, Parlin to mouth	0.3	1.8	1.6
Uncompahgre above Dallas Creek	4.5	27.6	24.7
Lower Gunnison	6.7	41.1	36.7
Naturita Creek near Norwood	2.8	17.2	15.3

TABLE III-18

SALT YIELDS FROM IRRIGATION IN SAN JUAN RIVER SUBBASIN (EPA, 1971)

	Average salt yield		
	(tons/acre/yr)	(kg/ha/day)	(lb/acre/day)
Fremont River above Torrey, Utah	0.4	2.5	2.2
Fremont River, Torrey to Hanksville, Utah	5.8	35.6	31.8
Muddy Creek above Hanksville, Utah	3.1	19.0	17.0
San Juan above Carracas	2.7	16.6	14.8
Florida, Los Pinos, Animas drainage	0.2	1.2	1.1
Lower Animas Basin	3.5	21.5	19.2
LaPlata River in Colorado	1.4	8.6	7.7
LaPlate River in New Mexico	0.3	1.8	1.6

TABLE III-19

SALT YIELDS FROM IRRIGATION IN LOWER COLORADO RIVER BASIN (EPA, 1971)

Area	Average salt yield		
	(tons/acre/yr)	(kg/ha/day)	(lb/acre/day)
Virginia River	2.3	14.1	12.6
Colorado River Indian Reservation	0.5	3.1	2.7
Palo Verde Irrigation District	2.1	12.9	11.5
Below Imperial Dam (Gila and Yuma projects)	variable	-	-

TABLE III-20

SALT YIELDS FROM IRRIGATION FOR SELECTED AREAS IN CALIFORNIA (WATER RESOURCES COUNCIL, 1971)

Area	Average salt yield		
	(tons/acre/yr)	(kg/ha/day)	(lb/acre/day)
North coastal	0.353	2.2	1.9
Central coastal	0.808	5.0	4.4
Sacramento	0.707	4.3	3.9
Delta-Central Sierra	0.974	6.0	5.3
San Joaquin	0.827	5.1	4.5
Tulare	0.768	4.7	4.2
Colorado Desert	10.9	67	60

3.8 POLLUTANTS FROM URBAN RUNOFF

From established urban areas, stormwater may pick up various wastes ranging from settled dust and ash to debris coming directly from man himself. The quantities of solids from urban nonpoint sources are quite significant in quantity. Fly ash and dust from industrial processes such as steel mills, cement manufacturing, and certain chemical processes are known to be profuse. Dusts from the burning of organic fuels are a significant factor, and solids in sizable quantities also result from off-street mud, automotive exhaust, organic debris from tree leaves and grass trimmings, and discarded litter.

In this report, the nonpoint pollutant loading function for urban areas is formulated from pollutant loading values obtained in a recent URS study (Amy, et al., 1974) for the U.S. Environmental Protection Agency. In that study URS reviewed a large number of published reports, extracted and statistically analyzed data, and presented average solid loading values and chemical and biological composition of solids.

In analyses of urban runoff data, URS assumed that only the runoff from street surfaces contributed to urban nonpoint pollution. The resulting loading values for solid wastes are given in terms of pounds per curb-mile per day. The user should note that these values represent contributions from both street and nonstreet surfaces.

Data developed in the URS study include nationwide means of solids loading rates and pollutant composition of street solids, as well as a more detailed breakdown of data into major source categories. Table III-21 shows data from the URS report which are divided into 13 subsets among three major source categories including climate, land use, and average daily traffic. These data may be different from the means which are given in the last column of the table, at the 80% confidence level. Whenever the mean of any parameter (solid loading rates or composition) in any subset differs significantly from the mean of the set of all data, that number may be substituted for the mean

TABLE III-21

SOLID LOADING RATES AND COMPOSITION--NATIONWIDE MEANS AND SUBSTITUTIONS OF THE NATIONWIDE MEANS AT 80% CONFIDENCE LEVEL* (AMY, ET AL., 1974)

		lbs/curb mi/day	Concentrations in micrograms per gram of dry solid														No./gram	
	Category	Loading	BOD_5	COD	OPO_4	NO_3	OrgN	Cd	Cr	Cu	Fe	Pb	Mn	Ni	Sr	Zn	TCOLI+	FCOLI+
Climate	Northeast	291_c					$5,970_c$	2.6_b	139_b		$17,700_b$	870_c	363_a	21_c	27_b	260_b		$4.4\Sigma5_c$
	Southeast	103_b	$29,100_b$		$2,240_a$		$1,970_a$			137_b		$1,370_b$		21_b	28_b			$7.0\Sigma4_d$
	Southwest	50_c			470_b				241_a	78_a		$2,520_b$		57_b	15_a		$5.7\Sigma6_d$	
	Northwest	30_c							246_a		$34,500_b$	$2,600_b$			10_c	480_a	$6.8\Sigma5_f$	$1.1\Sigma4_f$
Land Use	Openspace																	
	Residential		$14,000_b$	$82,000_b$	850_b	550_c	$1,800_a$			93_a		$1,430_b$		28_b				
	Commercial	74_c	$58,700_c$	$269,000_c$	$2,250_c$	$1,580_c$	6.430_a			133_b		$3,440_b$		48_b		520_b		
	Light Industry																	
	Heavy Industry								278_b		$28,600_b$	$1,160_c$	570_b				$8.2\Sigma5_e$	
Average Daily Traffic No./day	< 500											$1,210_d$				252_b		$6.9\Sigma4_f$
	500-5,000		$9,500_c$	$83,000_c$	741_d	419_b					$18,900_a$	$1,060_c$		17_d	34_c			$3.4\Sigma5_d$
	5,000-15,000														18_a			
	< 15,000	82_d											357_a				$3.8\Sigma5_a$	
	All data**	156_b	$19,900_b$	$140,000_b$	$1,280_b$	804_b	$2,950_b$	3.4_b	211_a	104_a	$22,000_a$	$1,810_a$	418_a	35_a	21_a	370_a	$2.5\Sigma6_c$	$1.7\Sigma5_b$

*Only those subset means are shown which differ from the mean of the set of all data at the 80-percent confidence level (Student $t > 1.39$. Degrees of Freedom ≥ 10). Total number of permitted substitutions = 103. Percent Standard Error of the Mean Subscripting Code: a=0-9, b=10-19, c=20-29, d=30-39, e=40-49, f=50-62.

+Coliform counts are expressed in computer notation, i.e. $\Sigma5=10^5$.

** Average TPO_4 is $2,930_c$ and NH_4 is $2,640_c$

of the set of all data. Table III-21 also gives the percent standard error of the mean which indicates the degree of confidence that may be placed on the mean.

Table III-22 presents the means and standard deviations of concentrations of mercury and several pesticides, which resulted from a set of data that were characterized as "very small and unreliable." No other data were obtained.

3.8.1 Loading Functions for Solids

$$Y(S)_u = L(S)_u \cdot L_{st} \qquad \text{(III-18)}$$

where

$Y(S)_u$ = daily total solids loading, kg/day (lb/day)

$L(S)_u$ = daily solids loading rates, kg/curb-km/day (lb/curb-mile/day)

L_{st} = steet curb-length (approximately 2.0 x street length), curb-km (curb-miles).

3.8.2 Loading Functions for Other Pollutants

$$Y(i)_u = a \cdot Y(S)_u \cdot C(i)_u \qquad \text{(III-19)}$$

where

$Y(i)_u$ = daily total loading of pollutant i, kg/day (lb/day); MPN(x 10^{-6}) per day for total coliform and fecal coliform.

a = conversion factor, 10^{-6} (metric and English).

$Y(S)_u$ = daily total loading of solids, kg/day (lb/day), calculated in Equation III-18.

$C(i)_u$ = concentration of pollutant i in solids, ug/g; MPN/g for total coliform and fecal coliform.

TABLE III-22

MEAN CONCENTRATIONS OF MERCURY AND CHLORINATED HYDROCARBONS IN STREET DIRT FROM NINE U.S. CITIES (AMY, ET AL., 1974)

	Concentrations in micrograms per kilogram of dry solid								
	Hg	Endrin	Dieldrin	PCB	Methoxy-chlor	DDT	Lindane	Methyl parathion	DDD
Mean	83	0.2	28	770	500	76	2.9	2	82
Standard deviation	111	-	28	770	1,050	118	7.1	-	78

Equations III-18 and III-19, along with solid loading values and compositions in Tables III-21 and III-22, provide the means to assess daily average pollutant loadings from urban areas.

It is important to note that pollutant loadings so calculated are street surface loadings rather than loadings at outfalls to the receiving waters. The transport of storm runoff in sewers and removal of pollutants in some treatment systems would reduce pollutant loads to some extent. Such effects are not included in loading factors suggested in Tables III-21 and III-22. Furthermore, the methodology presented above does not reflect the effect of housekeeping practices in the urban area. Good housekeeping practices such as cleaning of street solids by sweeping, and the use of catchment basins to remove solids and organic matter, will reduce pollutant loads from streets to receiving waters (Startor, 1972).

3.8.3 Procedure for Loading Calculations

Data in Tables III-21 and III-22 represent two options as well as two levels of accuracy for a user to assess pollutant loadings from a given urban area. Application of the "subset" data may result in higher accuracy, but require more data and more computation effort, than if "nationwide means" are used.

Option I - In this option the user will use nationwide means presented in Tables III-21 and III-22. Proceed as follows:

1. Determine solids loading rate and solids composition from tables.
2. Determine street length (include that of primary and secondary streets but not driveways, alleys, or parking lots).

3. Calculate daily solids loading using Equation III-18.

4. Calculate daily loading of other pollutants using Equation III-19.

Option II - In this option the user will make use of data presented for source categories in Table III-21. Steps needed for loading calculations are:

1. Characterize the study urban area. When applicable, the entire area should be divided into individual homogeneous sections with unique characteristics. Each individual section is then defined as a subarea (e.g., residential area).

2. Determine street length in each subarea.

3. Enter the Table III-21 at the line labeled "All Data."

4. Select a category of climate, land use, or average daily traffic, which best applies to an area and move upward to the line of data to the right of the category heading.

5. Substitute those values available in the row selected for the corresponding values in the row labeled "All Data." In choosing the substitute loading factors, the following priority sequence of source categories is suggested: (a) climate; (b) land use; (c) average daily traffic. The climatic zones of the U.S. delineated by the URS are shown in Figure III-18. Caution: it is not permissible to use more than one row of substitutions at a time, i.e., to use a BOD value for land use and COD for climate in order to form a new row of loading rate and composition data. It is both proper and useful, however, to repeat the above process to obtain several new rows of data to

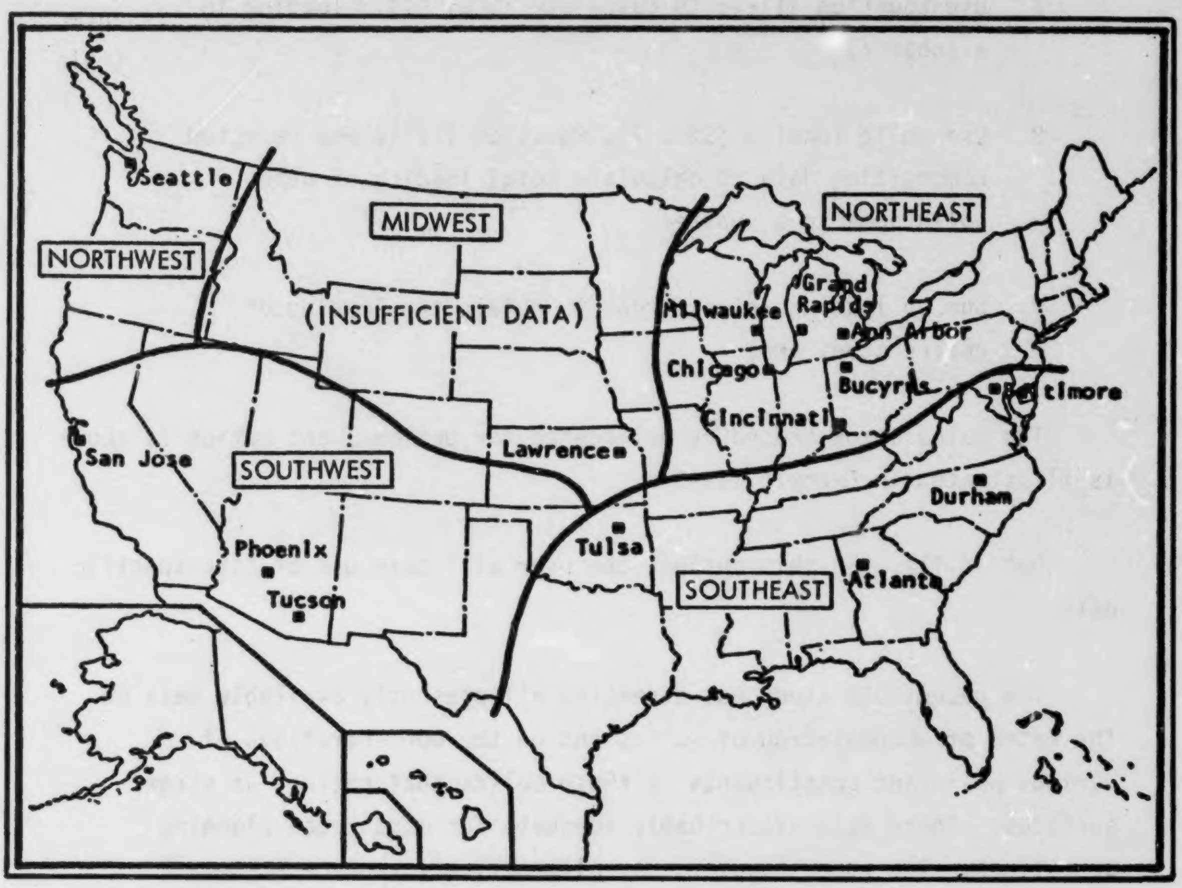
Seattle
MIDWEST
NORTHEAST
NORTHWEST
(INSUFFICIENT DATA)
Grand Rapids
Milwaukee
Ann Arbor
Chicago
Bucyrus
Baltimore
Cincinnati
San Jose
SOUTHWEST
Lawrence
Durham
Phoenix
Tulsa
SOUTHEAST
Atlanta
Tucson

FIGURE III-18 CLIMATE ZONE FOR THE CITIES FROM WHICH DATA ARE AVAILABLE AND USED IN THE URS STUDY (AMY, ET AL., 1974)

present a range of composition and loading rates. Use data from Table III-22 if desired.

6. Repeat Steps 4 and 5 for all subareas.

7. Use Equation III-18 to calculate total solid loading in a subarea.

8. Use solid loading (Step 7), Equation III-19 and selected composition data to calculate total loading of other pollutants in a subarea.

9. Sum up loadings of subareas to obtain the loading of entire study area.

The calculation procedure delineated for Option I and Option II above is illustrated in Example III-5.

Option III - In this option, the user will make use of site specific data.

The recent URS study has assembled all presently available data on the rates of accumulation of solids and on the concentrations of various pollutant constituents in those solids that collect on street surfaces. These data are probably adequate for most urban planning operations. The user, however, may alternatively replace these loading factors by site specific data to obtain better prediction.

If site specific data are lacking, users are encouraged to conduct sampling and analytical programs of their own. The data from site specific tests, if handled properly, may be used in analyzing the area's runoff problems instead of using values given in this report. This would be desirable in most instances, especially in areas or under specific conditions that were not documented in the URS study.

Recommended procedures for conducting site specific tests are given in Appendix B of the URS Report (Amy, et al., 1974).

With the lack of site specific data, the user may wish to examine the available published data for source and reliability. The user is referred to Appendix A of the URS Report for description of available data sources, as well as procedures for processing these data.

3.8.4 Street Length and Land Use Data for Urban Areas

Street length data are available from local public works departments or street departments. They can also be obtained by measurement on aerial photographs.

Survey statistics for the U.S. indicate that street surfaces occupy on the average about one-sixth of the urban area (Manuel, et al., 1968). The American Public Works Association (American Public Works Association, 1974) recently developed a regression relationship between curb length of urban area versus population density. Data from many cities across the country were used. The resulting regression equation is:

$$CL = 413.11 - (352.66)(0.839)^{PD} \qquad \text{(III-20)}$$

where

CL = curb length density, ft/acre

PD = population density, number/acre

The correlation coefficient for the equation is 0.72. The regression curve is shown in Figure III-19.

Curb length can be estimated if street surface acreage is known. Table III-23 presents equivalent curb length per unit area of street surface, suggested by URS. However, if actual values are known, it is best to use known values.

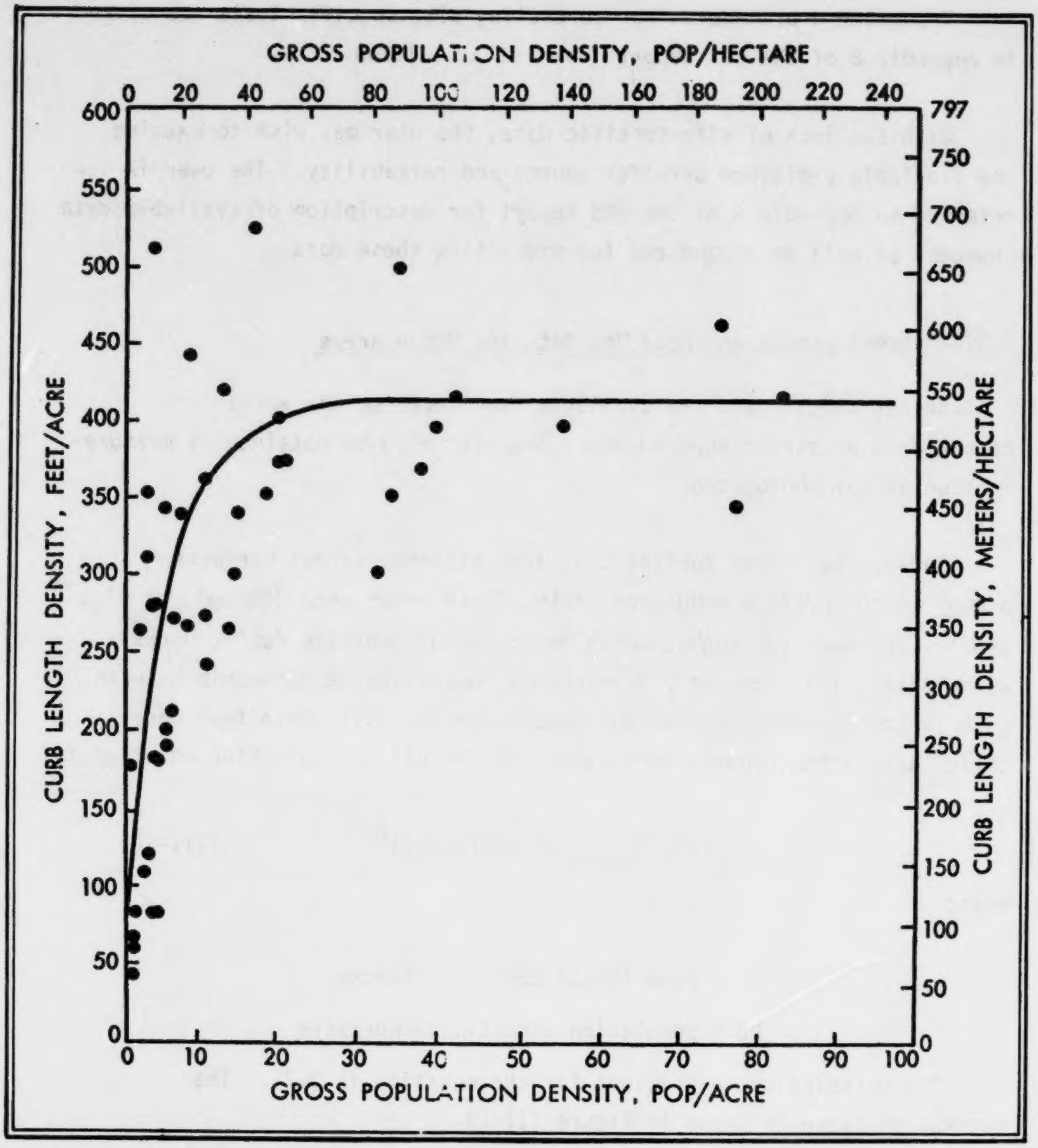

FIGURE III-19 CORRELATION BETWEEN POPULATION DENSITY AND CURB LENGTH DENSITY. (AMERICAN PUBLIC WORKS ASSOCIATION, 1975)

TABLE III-23

EQUIVALENT CURB-LENGTH PER UNIT AREA OF STREET SURFACE, ARRANGED BY LAND USE TYPES (AMY, ET AL., 1974)

	Equivalent curb-km per hectare of street surface	Equivalent curb-miles per acre of street surface
Open land	2.11	0.53
General residential	2.15	0.54
General commercial	1.63	0.41
Light industrial	1.71	0.43
Heavy industrial	1.59	0.40
All land use types	1.83	0.46

TABLE III-24

GENERAL LAND CONSUMPTION RATES FOR VARIOUS LAND USES (AMERICAN PUBLIC WORKS ASSOCIATION, 1974)

	Land consumption (acres/capita)		
Land Use	<100,000 Population	>100,000 Population	>250,000 Population
Residential	0.1049	0.0714	0.0585
Commercial	0.0101	0.0084	0.0073
Industrial	0.0177	0.0083	0.0077
Park	0.0146	0.0093	0.0078

The following references provide survey data and analysis results relative to land uses in major urban areas of the U.S.:

Bartholomew, H., Land Use In American Cities, Harvard University Press, Cambridge, Massachusetts (1955).

Niedercorn, J. H., and E. F. R. Hearle, "Recent Land-Use Trends in Forty-Eight Large American Cities," The RAND Corporation, Santa Monica, California, Memorandum RM-3664-FF, June 1963.

Manuel, A. D., R. H. Gustafson, and R. B. Welch, "Three Land Research Studies," National Commission on Urban Problems, Research Report No. 12, Washington, D.C. (1968).

The American Public Works Association (1974) estimated land consumption rates for various land uses in American cities, shown in Table III-24. These rates can be used to estimate acreages in different land uses if the number of population is known.

EXAMPLE III-5

The study area is a 250-acre urban watershed in Atlanta, Georgia. The area is mainly residential and has 17 curb-miles of primary and secondary streets. Predict the average daily loadings of BOD and lead in runoff from the entire area.

Option I - Use nationwide means of solid loading rate and compositions given in Table III-21.

Calculate solid loading:

$$L(S)_u = 156 \text{ lb/curb-mile/day}$$

$$Y(S)_u = 156 \cdot 17 = 2{,}652 \text{ lb/day}$$

Calculate BOD and Pb loadings:

$$C(BOD)_u = 19{,}900 \cdot 10^{-6} \text{ lb/lb solid}$$

$$C(Pb)_u = 1{,}810 \cdot 10^{-6} \text{ lb/lb solid}$$

$$Y(BOD)_u = 2{,}652 \cdot 19{,}900 \cdot 10^{-6} = 52.8 \text{ lb/day}$$

$$Y(Pb)_u = 2{,}652 \cdot 1{,}810 \cdot 10^{-6} = 4.8 \text{ lb/day}$$

Option II - Use substitutions at 80% confidence level.

Atlanta is in the southeast. Move upward in Table III-21 to southeast climate category. A loading substitution is available. Make all available substitutions into the row labeled "All Data." The new row has, among others:

Solid loading rate, $L(S)_u = 103$ lb/curb-mile/day

BOD concentration, $C(BOD)_u = 19{,}900 \cdot 10^{-6}$ lb/lb solid

Pb concentration, $C(Pb)_u = 1{,}370 \cdot 10^{-6}$ lb/lb solid

Calculate solid loading:

$$Y(S)_u = 103 \cdot 17 = 1{,}751 \text{ lb/day}$$

Calculate BOD and Pb loadings:

$$Y(BOD)_u = 1{,}751 \cdot 19{,}900 \cdot 10^{-6} = 34.8 \text{ lb/day}$$

$$Y(Pb)_u = 1{,}751 \cdot 1{,}370 \cdot 10^{-6} = 2.40 \text{ lb/day}$$

Pollutant loadings calculated in Option II are lower than those in Option I and probably better represent the real situation in Atlanta, Georgia.

END OF EXAMPLE III-5

3.9 POINT SOURCE WASTE LOADS

3.9.1 General

The purpose of this section is to discuss sources of information concerning point source discharges and to provide a reasonable range of values for discharge concentrations when no direct source data can be located. When available, direct data concerning an existing or proposed discharge is preferable to the use of the information presented here.

The point-source information presented here is divided into three catergories: municipal discharges, urban storm water discharges, and industrial discharges. Three basic sources of information were used in collecting the data: EPA guidelines for future discharges, the results of recent attempts (e.g., Tetra Tech, Kaiser Engineers) to characterize point source waste loads, and data from a number of actual continuous flow discharges. These types of information are identified as they appear in the subsequent tables.

3.9.2 Sources of Direct Data

Before using these guidelines and estimates a planner should exhaust the sources of actual point source waste loading information available to him. The discharger may be the best source of information since many states require dischargers to maintain a self monitoring program. Pollutant load per day and pollutant concentration data are usually included in this information. Second, essentially all point source discharges are now required to obtain a discharge permit. The state or federal agency issuing these permits will have on file maximum allowable limits for the discharge. These limits can be used as an upper bound waste loading rate. Third, state water quality or water resource agencies often have conducted sample collection programs for significant discharges. Fourth, data for a similar facility within the local region (same activity, same general size) may be used as an estimate for an unknown waste load. If none of these are available the data presented here may be used as a reasonable "ball park" estimate.

3.9.3 Municipal Discharges

Total municipal discharge loads show a strong connectivity with population levels. Additionally, municipal discharge concentrations are reasonably consistant from one municipality to another. Thus municipal waste load estimates presented here represent better estimates for a specific discharge than do urban stormwater or industrial discharges.

Table III-25 lists typical concentrations of basic water quality parameters which generally concern the planner. This table is based on assessment of a large number of individual discharges. Based on the existing level of a specific municipal wastewater treatment system, one of the three columns in the table could be used to approximate municipal waste loadings. The breath of variety which exists in actual municipal discharges is shown in Tables III-26a and III-26b where fifteen municipal discharges and six discharges in a designated 208 area in North Carolina are characterized, respectively. Table III-26b also indicates relative treatment levels by listing both inflow and outflow concentrations.

Table III-27 lists average daily flow rates for various contributors to municipal systems on a per capita basis. Multiplying this per capita flow rate times the municipal waste load concentrations (from Table III-25) produces a waste load per capita per day. Using this figure either total present municipal waste loading (population multiplied by waste load per person per day) or future municipal waste loading increases (projected population increase multiplied by waste load per person per day) may be estimated.

Finally, Table III-28 shows major pollutant effluent concentrations which conform to BAT treatment (1983 standard). Comparison of this

TABLE III-25

TYPICAL MUNICIPAL WASTE CONCENTRATIONS (METCALF AND EDDY, 1972)

Constituent	Concentration mg/l		
	Strong	Medium**	Weak
Solids, total	1,200	700	350
Dissolved, total	850	500	250
Fixed	525	300	145
Volatile	325	200	105
Suspended, total	350	200	100
Fixed	75	50	30
Volatile	275	150	70
Settlable solids, (ml/liter)	20	10	5
Biochemical oxygen demand, 5-day, 20°C (BOD_5-20°)	300	200	100
Total organic carbon (TOC)	300	200	100
Chemical oxygen demand (COD)	1,000	500	250
Nitrogen, (total as N)	85	40	20
Organic	35	15	8
Free ammonia	50	25	12
Nitrites	0	0	0
Nitrates	0	0	0
Phosphorus (total as P)	20	10	6
Organic	5	3	2
Inorganic	15	7	4
Chlorides*	100	50	30
Alkalinity (as $CaCO_3$)*	200	100	50
Grease	150	100	50

*Values should be increased by amount in carriage water.
**In the absence of other data use medium strength data for planning purposes.

TABLE III-26a
EXAMPLES OF MUNICIPAL DISCHARGES

Discharge	Flow Rate Mgd	BOD mg/1	BOD g/cap/day	TSS mg/1	TDS mg/1	Turbidity JTU	Total N mg/1	Total P mg/1	COD mg/1
Los Angeles City[2], Ca.	350.4	320	--	330	--	7.1	107	52	610
Los Angeles Hyperion[2],Ca	332.2	113	--	85.7	--	54	17.3	20	240
Huntington Bch.[2], Ca.	154.7	180	--	96.4	2611	100	48.2	--	646
San Diego[2], Ca.	100.1	172	--	134	1986	256	42	--	626
Terminal Island[2], Ca.	9.1	165	--	135	--	--	33	10	--
Vallejo San. Dst.[3], Ca.	5.3	124	31.3	--	--	--	32	47	336
Marin County #5[3], Ca.	0.4	91	28.5	--	--	--	30	27	272
Richmond (City)[3], Ca.	6.2	138	50.3	--	--	--	23	23	435
San Francisco[3], Ca.	47.9	158	69.9	--	--	--	33	21	389
San Francisco S.E.[3], Ca	19.1	158	71.2	--	--	--	33	21	445
Belmont[3], Ca.	3.5	87	28.8	--	--	--	22	24	280
Petaluma[3], Ca.	1.0	36	9.1	--	--	--	28	55	190
Islip[5], N.Y.	0.168	29.3	--	48	--	--	9.1	--	--
Huntington[5], N.Y.	1.77	29.8	--	32	--	--	18.1	5.3	--
Port Jefferson[5], N.Y.	1.44	68.8	--	54.2	--	--	19.4	5.9	--

1. Based on systems receiving primary treatment
2. Tetra Tech, Inc., 1975
3. Pearson, Storrs and Selleck, 1969
4. All data is for one year only and is designed to indicate a range of probable values.
5. Weston, 1976.

TABLE III-26b

POINT SOURCE LOADINGS OF SIX MAJOR WASTEWATER TREATMENT FACILITIES IN ONE NORTH CAROLINA 208 AREA

FACILITY NAME	FLOW (MGD)	BOD (lb/day)		S.S. (lb/day)		Sludge Production
		INFLUENT	EFFLUENT	INFLUENT	EFFLUENT	(lb/day Dry Solids)
Northside	8.4	16,673	1,331	13,731	2,522	4,300
Third Fork	3.4	7,231	993	5,189	2,410	3,300
New Hope	3.4	5,473	1,758	4,197	1,361	318
Chapel Hill	3.8	5,166	824	5,071	1,363	3,000
Walnut Creek	20.3	64,673	11,005	23,702	7,111	6,255
Hillsborough	0.56	957	383	934	234	0 - See Text
SUB-TOTAL	39.9	100,173	16,294	52,824	15,001	17,173
Percent of TOTAL	84.8%	90.7%	95.3%	84.4%	93.0%	

TABLE III-27

MUNICIPAL WASTEWATER FLOW RATES

Category	Waste Flow Per Day
Domestic - Single Dwelling[1]	67 gcd[3]
Domestic - Multiple Dwelling[1]	55 gcd
Industrial - Major Cannery Activity[1]	500 gmd[4]
Industrial - Diversified[1]	400 gmd
Industrial - Minimal Activity[1]	175 gmd
Commercial Activity[1]	100 gcd[5]
Total Municipal System[2]	140 - 170 gcd

[1] Tetra Tech, Inc. 1975

[2] Metcalf and Eddy, 1972

[3] gcd = gallons per capita per day

[4] gmd = gallons per manufacturing employee per day

[5] ged = gallons per commercial employee per day

TABLE III-28

MUNICIPAL EFFLUENT CONCENTRATIONS CORRESPONDING TO EPA BEST AVAILABLE TECHNOLOGY (BAT) TREATMENT LEVELS (TETRA TECH, INC. 1975)

Constituent	BAT Concentration (mg/l)
BOD	10
TSS	10
Total N	5
Total P	3
Grease	3

data with data on influent concentrations in Table III-25 indicates the degree of treatment necessary to conform to these standards.

3.9.4 Urban Stormwater Waste Load Estimation

Stormwater waste loads enter a natural water body either through a storm sewer system or through a combined sewer system. Typical values of pollutant concentrations for the stormwater overflow through each of these systems is shown in Table III-29. Tables III-30 and III-31 show data collected for a number of actual storm sewer and combined systems respectively. Finally, a summary table of pollutant loading for stormwater discharges is shown in Table III-32.

3.9.5 Industrial Waste Loadings

Industrial waste loadings are difficult to accurately assess from either national averages or published guidelines. Both flow rate and pollutant concentrations will vary substantially as a function of plant size, processes involved, primary and secondary product mix, etc. Waste discharge data for a number of individual plants is provided in Tables III-33, III-34a and b, and III-35. Plants within a local region which are similar to any of the listed plants may be used as a guide to local plant discharge concentrations. These data are further generalized in Tables III-36 and III-37 which report Best Practical Technology (BPT) and Best Available Technology (BAT) guidelines for six types of industrial activities. Individual processes for these industries will have different guidelines. Table III-38 defines BAT guidelines for selected parameters and industrial processes.

TABLE III-29

CHARACTERISTIC VALUES FOR URBAN STORMWATER AND SEWER OVERFLOW WATER QUALITY (FIELD, 1975)

Water Quality Parameter	Combined Sewer Overflow Characteristic Concentration (mg/l)	Urban Stormwater Characteristic Concentration (mg/l)
BOD_5	100 - 500	11 - 500
TSS	100 - 1500	500 - 11,300
Total Solid	300 - 2000	1000 - 14,600
Organic N	3.5 - 30.1	0.9 - 16
NH_3N	1.1 - 11.5	0.4 - 2.5
PO_4	1 - 62	10 - 125
Total coliform	5×10^4 - 30×10^6 MPN/ml	2×10^3 - 14×10^5
pH	4.9 - 8.4	*
Fecal coliform	5×10^4 - 11×10^6 MPN/ml	
Chloride		200 - 25,000
Oils		10 - 110
Phenols		0 - 0.2
Lead		0 - 1.9

*No data available

TABLE III-30
COMPARISON OF QUALITY OF STORM SEWER DISCHARGES FOR VARIOUS CITIES[a]
(LAGER AND SMITH, 1974)

Type of wastewater, location, year, Ref. No.	BOD_5, mg/l		COD, mg/l		DO, mg/l	SS, mg/l		Total coliforms, MPN/100 ml		Total nitrogen, mg/l as N	Total phosphorus, mg/l as P
	Avg	Range	Avg	Range	Avg	Avg	Range	Avg	Range	Avg	Avg
Typical untreated municipal	200	100-300	500	250-750	--	200	100-350	$5x10^7$	$1x10^7$-$1x10^9$	40	10
Typical treated municipal											
Primary effluent	135	70-200	330	165-500	--	80	40-120	$2x10^7$	$5x10^6$-$5x10^8$	35	7.5
Secondary effluent	25	15-45	55	25-80	--	15	10-30	$1x10^3$	$1x10^2$-$1x10^4$	30	5.0
Storm sewer discharges											
Ann Arbor, Mich., 1965 (2)	28	11-62	--	--	--	2,080	650-11,900	--	--	3.5	1.7
Castro Valley, Calif., 1971-72 (14)	14	4-37	--	--	8.4	--	--	$2x10^4$	$4x10^3$-$6x10^4$	1.9[b]	--
Des Moines, Iowa, 1969 (6)	36	12-100	--	--	--	505	95-1,053	--	--	2.2	0.87
Durham, N.C., 1968 (1)	31	2-232	224	40-660	--	--	--	$3x10^5$	$3x10^3$-$2x10^6$[c]	--	0.18
Los Angeles, Calif., 1967-68 (19)	9.4	--	--	--	6.9	1,013	--	--	$3x10^3$-$2x10^6$	--	--
Madison, Wis., 1970-71 (17)	--	--	--	--	--	81	10-1,000	--	--	4.8	1.1
New Orleans, La., 1967-69[d] (16)	12	--	--	--	4.5	26	--	$1x10^6$	$7x10^3$-$7x10^8$	--	--
Roanoke, Va., 1969 (12)	7	--	--	--	--	30	--	--	--	--	--
Sacramento, Calif., 1968-69 (37)	106	24-283	58	21-176	--	71	3,211	$8x10^5$	$2x10^4$-$1x10^7$[c]	--	--
Tulsa, Okla., 1968-69 (33)	11	1-39	85	12-405	--	247	84-2,052	$1x10^5$	$1x10^3$-$5x10^8$	0.3-1.5[e]	0.2-1.2[f]
Washington, D.C., 1969 (5)	19	3-90	335	29-1,514	--	1,697	130-11,280	$6x10^5$	$1x10^5$-$3x10^6$	2.1	0.4

---

a. Data presented here are for general comparisons only. Since different sampling methods, number of samples, and other procedures were used, the reader should consult the references before using the data for specific planning purposes.

b. Only ammonia plus nitrate.

c. Only fecal.

d. Median values from 1 sampling station.

e. Only organic (Kjeldahl) nitrogen.

f. Only soluble orthophosphate.

TABLE III-31

COMPARISON OF QUALITY OF COMBINED SEWAGE FOR VARIOUS CITIES[a], (LAGER AND SMITH, 1974)

Type of wastewater, location, year, Ref. No.	BOD_5, mg/l		COD mg/l		DO, mg/l	SS mg/l		Total coliforms, MPN/100 ml		Total nitrogen, mg/l as N	Total phosphorus, mg/l as P
	Avg	Range	Avg	Range	Avg	Avg	Range	Avg	Range	Avg	Avg
Typical untreated municipal	200	100-300	500	250-750	--	200	100-350	$5x10^7$	$1x10^7$-$1x10^9$	40	10
Typical treated municipal											
Primary effluent	136	70-200	330	165-500	--	80	40-120	$2x10^6$	$5x10^6$-$5x10^8$	35	7.5
Secondary effluent	25	15-45	55	25-80	--	15	10-30	$1x10^3$	$1x10^2$-$1x10^4$	30	5.0
Selected combined											
Atlanta, Ga., 1969 (31)	100	48-540	--	--	8.5	--	--	$1x10^7$	--	--	1.2[b]
Berkeley, Calif. 1968-69 (34)[c]	60	18-300	200	20-600	--	100	40-150	--	--	--	--
Brooklyn, N.Y., 1972 (8)	180	86-428	--	--	--	1,051	132-8,759	--	--	--	1.2[b]
Bucyrus, Ohio 1968-69 (35)	120	11-560	400	13-920	--	470	20-2,440	$1x10^7$	$2x10^5$-$5x10^7$	13	3.5
Cincinnati, Ohio, 1970 (36)	200	80-380	250	190-410	--	1,100	500-1,800	--	--	--	--
Des Moines, Iowa, 1968-69 (6)	115	29-158	--	--	--	295	155-1,166	--	--	12.7	11.6
Detroit, Mich., 1965 (2)	153	74-685	115	--	--	274	120-804	--	--	16.3[d]	4.3
Kenosha, Wis., 1970 (18)	129	--	464	--	--	458	--	$2x10^6$	--	10.4[d]	5.9
Milwaukee, Wis., 1969 (7)	55	26-182	177	118-765	--	244	113-848	--	$2x10^5$-$3x10^7$	3-24	0.8[b]
Northampton, U.K., 1960-62 (22)	150	80-350	--	--	--	400	200-800	--	--	10[c]	--
Racine, Wis., 1971 (18)	119	--	--	--	--	439	--	--	--	--	--
Roanoke, Va., 1969 (12)	115	--	--	--	--	78	--	$7x10^7$	--	--	--
Sacramento, Calif., 1968-69 (37)	165	70-328	238	59-513	--	125	56-502	$5x10^6$	$7x10^5$-$9x10^7$[g]	--	--
San Francisco, Calif., 1969-70 (3)	49	1.5-202	155	17-626	--	68	4-426	$3x10^6$	$2x10^4$-$2x10^7$	--	--
Washington, D.C., 1969 (5)	71	10-470	362	80-1,760	--	622	35-2,000	$3x10^6$	$4x10^5$-$6x10^6$	3.5	1.0

---

a. Data presented here are for general comparisons only. Since different sampling methods, number of samples, and other procedures were used, the reader should consult the references before using the data for specific planning purposes.

b. Only orthophosphate.

c. Infiltrated sanitary sewer overflow.

d. Only ammonia plus organic nitrogen (total) Kjeldahl).

e. Only ammonia.

f. Only fecal.

TABLE III-32

SUMMARY OF STORMWATER POLLUTANT CONCENTRATIONS (KAISER ENGINEERS, 1969)

	Stormwater Overflow Concentrations			
	Separate Drainage Areas[a]		Combined Areas[b]	
Pollutant[c]	Mean	Standard Deviation	Mean	Standard Deviation
BOD_5	27	25	108	36
COD	205	118	284	110
S.S.	608	616	372	275
Total Coliforms[d]	$3x10^5$	-	$6x10^6$	-
Total Nitrogen (as N)	2.3	1.4	9	6
Total Phosphorus (as P)	0.5	0.4	2.8	2.9

(a) Summary of 20 cities, storm sewers and unsewered areas

(b) Summary of 25 cities, combined sewer areas

(c) All units mg/l except coliforms, MPN/100 ml

(d) Geometric mean

TABLE III-33

TYPICAL INDUSTRIAL DISCHARGE POLLUTANT CONCENTRATIONS

Industry	Flow Rates mg/1000 lb[1]	BOD lb/1000 lb	COD lb/1000 lb	TSS lb/1000 lb	Total N lb/1000 lb	Total P lb/1000 lb	Heavy Metal lb/1000 lb	Oil & Grease lb/1000 lb
Primary Metal[2]	0.2-1.6	-	-	-	32	15	55-242	-
Cu, Brass Rods[3]	0.04	0.1	0.5	0.02	0.07	~0	-	-
Roofing Materials[3]	0.01	13.6	25.7	14.2	0.34	~0	0.13	0.68
Steel Plate, Wire[3]	0.004	0.56	2.7	5.1	0.04	0.01	1.2	0.12
Petroleum (General)[2]	0.005	1.3	3.7	-	0.4	-	0.003	0.15
Oil Production(#1)[3]	0.003	0.57	2.1	0.58	0.33	0.01	0.05	0.14
Oil Production(#2)[3]	0.003	0.45	1.3	0.86	0.16	0.01	0.03	0.09
Oil Production(#3)[3]	0.001	0.45	2.9	0.65	0.24	0.01	0.04	0.31
Paper (General)[2]	0.015	18	55	28	-	-	-	-
Paperboard[3]	0.017	19.6	64.8	37.9	0.05	-	-	-
Paper[3]	0.024	12.6	43	33.1	0.03	-	-	-
Primary Inorganic[2]	0.002	0.2-3.5	-	5-30	0.03-0.7	0.8-9.0	0.05-0.3	0.06-2
Alky Lead Flugro Hydrocarbons[3]	0.002	0.39	0.89	0.15	0.03	-	-	0.1
Inorg. Acids[3]	0.004	0.08	0.52	5.37	0.04	0.02	0.33	0.06
Primary Organic[2]	0.002	1-1.9	-	-	3-7	0.15-0.3	0.01-0.02	0.05-0.08
Caustic Chemicals[3]	0.021	1.24	4.9	19.9	1.27	0.3	1.69	0.24
Plant Food[3]	0.001	0.03	1.43	0.01	1.17	0.14	0.02	0.02

1 Units are million gallons of pollutant per 1000 lb. of finished product

2 Kaiser Engineers, 1969

3 Pearson, Storrs, Sellech, 1969

TABLE III-34a

SELECTED SAN FRANCISCO AREA INDUSTRIAL DISCHARGERS

DISCHARGER	FLOW	BOD		SS		OIL & GREASE		ORGANIC NITROGEN		TOTAL PHOSPHATE		TOTAL COLIFORM	pH	LEAD		ZINC		PHENOLIC COMPOUNDS	
	MGC	Conc*	Load**	Conc	Load	Conc	Load	Conc	Load	Conc	Load	MPN Per 100 ml		Conc	Load	Conc	Load	Conc	Load
Shell Oil	3.4	9	250	28	800	15	430	NR	-	NR	-	< 100	7.0	NR	-	80	2	20	0.6
C&H Sugar	21	51	9000	3	470	2	320	NR	-	NR	-	NR	7.8	NR	-	NR	-	NR	-
Hercules	1.6	NR	-	30	380	3	37	5	62	NR	-	< 10	-	NR	-	NR	-	NR	-
	0.2	9	17	50	95	3	5	5	9	NR	-	< 100	7.6	NR	-	NR	-	NR	-
Standard Oil	109	9	8200	14	13,000	3	2700	NR	-	NR	-	NR	-	NR	-	20	22	14	13
Stauffer Co.	0.1	NR	-	5	5	4	4	NR	-	NR	-	NR	-	30	0.02	NR	-	NR	-
Stauffer Co.	0.08	NR	-	7	4	2	1	NR	-	NR	-	NR	-	-	-	-	-	NR	-
	36.5	NR	-	Net Loss	-	NR	-	1	300	1	300	NR	7.8	Net Loss	-	180	56	NR	-
U.S. Steel	22	NR	-	6	1,200	1	200	NR	-	-	-	NR	-	10	2	120	2	30	5
DeLavol Turbine	0.6	15	75	14	70	0.6	3	NR	-	NR	-	NR	7.0	NR	-	NR	-	NR	-
	0.15	3	4	7	9	1	1	NR	-	NR	-	NR	7.0	NR	-	NR	-	10	0.01
N. Elec.	0.16	7	9	3	4	1	2	NR	-	NR	-	< 10	-	9	0.01	160	0.2	NR	-

*Conc = Concentration in mg/l

**Load = Load in lb/day

TABLE III-34b
SELECTED SAN FRANCISCO AREA INDUSTRIAL DISCHARGERS

DISCHARGER	FLOW	BOD		SS		OIL & GREASE		ORGANIC NITROGEN		TOTAL PHOSPHATE		TOTAL COLIFORM	pH	LEAD		ZINC		PHENOLIC COMPOUNDS	
	MGD	Conc*	Load**	Conc	Load	Conc	Load	Conc	Load	Conc	Load	MPH Per 100 ml		Conc	Load	Conc	Load	Conc	Load
Allied Chemical	3.3	5	140	Net Loss	-	-	-	NR	-	NR	-	NR	7.8	100	3	60	1.5	NR	-
Dow Chemical	22.2	2		28		-	-	NR	-	NR	-	NR	7.3	7	1.3	NR	-	NR	-
Exxon Oil	2.3	24	400	23	300	1	20	NR	-	NR	-	NR	7.2	NR	-	20	0.4	190	2.5
FMC	0.7	NR	-	11	75	NR	-	NR	-	26	160	NR	7.6	NR	-	NR	-	NR	-
Marine World	1.4	2	15	Net Loss	-	3	35	1	15	0.5	7	7000	7.8	NR	-	NR	-	NR	-
Merck Co.	3.2	NR	-	740	21,000	18	530	NR	-	NR	-	NR	10.1	NR	-	NR	-	NR	-
G&E, #1	0.3	NR	-	75	200	1	3	NR	-	NR	-	NR	9.0	NR	-	NR	-	NR	-
	0.03	NR	-	0.6	2	NR	-	NR	-	NR	-	NR	11	NR	-	NR	-	NR	-
G&E, #2	12.2	NR	-	75	8,000	1	85	NR	-	NR	-	NR	7.8	5	0.5	50	5	NR	-
	0.005	NR	-	0.3	0.01	2	0.1	NR	-	NR	-	NR	9.9	NR	-	NR	-	NR	-
Sequoia Oil	0.1	13	9	47	40	8	7	NR	-	NR	-	NR	7.8	NR	-	70	0.05	40	0.04
Phillips Oil	10.5	9	800	33	3,000	2	180	NR	-	NR	-	<100	-	-	-	20	2	10	1
S.F. Airport	0.8	28	190	82	420	2	11	NR	-	NR	-	<100	-	10	0.06	120	0.7	7	0.04

*Conc = Concentration in mg/l

**Load = Load in lb/day

TABLE III-35

SUMMARY OF CURRENT AND PROJECTED WASTE LOADS IN ONE REGION 208 AREA (BY SIC)

SIC GROUP		CURRENT LOADINGS		BEST PRACTICABLE WASTE REDUCTION TECHNOLOGY			PROJECTED LOADINGS	
		BOD (lb/day)	SS (lb/day)		Expected Reductions		BOD (lb/day)	SS (lb/day)
No.	Description	Sewer	Sewer	Description	BOD(%)	SS(%)	Sewer	Sewer
201	Meat Products	1,523	1,059	Anaerobic Lagoon to Stabilization Pond	90	85	152	117
202	Dairy Products	973	400	Anaerobic Digestion & Clarification	85	90	71	40
204	Grain Mill Prods.	180	50	Oxidation Ditch & Clarification	85	75	27	13
205	Bakery Prods.	935	910	Rotating Bio-Filters & Clarification	85	65	140	319
208	Soft Drinks	330	40	Fixed Activated Sludge	84	65	53	14
211	Tobacco Man.	2,024	1,750	Activated Sludge (E.A.) & Clarification	85	75	304	438
22_	Textile Mill	2,530	2,173	Activated Sludge & Alum-Aided Clarif.	85	75	380	543
226	Dyeing & Fin.	0	0	Carbon Adsorption & Clarification	75	60		
251	Furniture	0	0	--	--	--		
265	Paperboard Con.	245	150	Screening, Ext. Aeration, Clarification	35	65	159	53
27_	Print. & Pub.	0	0	--	--	--		
28_	Chem. & Allied P.	64	29	Activated Sludge & Clarification	85	75	10	18
32_	Stone, Clay P.	0	0	Stilling Ponds, Water Recycle	30	70		
35_	Machinery.	32	79	Oil & Grease Traps	50	65	16	28
36_	Elect. Equip.	659	402	Ion Exchange (for Plating Process)	10	90	593	40
379	Transp. Equip.	100	100	Oil & Grease Traps	50	65	50	50
---	Non-Manuf.	1,374	170	See Text	70	90	412	17
9999	Mun. W.W.T.P.	0	0	Upgrade Six Largest Plants	Varies for Each Plant			
	TOTALS	10,469	7,312	--	--		2,367	1,690

TABLE III-36

TYPICAL INDUSTRIAL EFFLUENT CONCENTRATIONS
BPT - "1977"
APPROXIMATE MEAN EFFLUENT CHARACTERISTICS(a) (EPA, 1974)
(in mg/l)

Parameter	20 Food	22 Textiles	26 Paper	28 Chemicals	29 Petroleum	33 Metals
TSS	40(b)	49(b)	58(b)	40(b)	10(b)	20(b)
BOD_5	29(b)	22(b)	39(b)	30(b)	15(b)	38(b)
COD	135	225(b)	156	1400(b)	102(b)	-
TDS	-	700	3785	4350	-	-
Cl	565	25	135	-		-
Total-P	17	2	-	5		.50
Total-N	50	2	-	20	70	62(b)
Lead	-	.03	-	2(b)		.25(b)
Zinc	-	5	-	0.25(b)		.25(b)
Cadmium	-	.005	-	0.225(b)		.15(b)
Oil	10(b)	10	-	15(b)	5(b)	5(b)

(a) Represents an approximate estimate of the mean effluent concentrations for each industry. The pollutant concentration could vary widely within an industry as a result of varying water usages.

(b) Concentrations developed from effluent guidelines which exist for these parameters, other concentrations were obtained from existing treatment plant data.

TABLE III-37

TYPICAL INDUSTRIAL EFFLUENT CONCENTRATIONS
BAT - "1983"
APPROXIMATE MEAN EFFLUENT CHARACTERISTICS(a) (EPA, 1974)
(in mg/l)

Parameter	20 Food	22 Textiles	26 Paper	28 Chemicals	29 Petroleum	33 Metals
TSS	10(b)	8(b)	23(b)	13(b)	5(b)	5(b)
BOD$_5$	7(b)	11(b)	22(b)	15(b)	5(b)	4(b)
COD	48	72(b)	88	460(b)	26(b)	-
TDS	-	700	3785	4350	-	-
Cl	565	25	135	-	-	-
Total-P	1.2	2	-	3	-	0
Total-N	9(b)	2	-	3	28	5(b)
Lead	-	.03	-	1(b)	-	0(b)
Zinc	-	5	-	.25(b)	-	0(b)
Cadmium	-	.005	-	.05(b)	-	0(b)
Oil	10(b)	10	-	3	1.35(b)	5(b)

(a) Represents an approximate estimate of the mean effluent concentrations for each industry. The pollutant concentration could vary widely within an industry as a result of varying water usages.

(b) Concentrations developed from effluent guidelines which exist for these parameters, other concentrations were obtained from any existing treatment plant data found in the EPA Development Documents (2-14).

TABLE III-38

EPA BAT GUIDELINES FOR SELECTED NEW SOURCE INDUSTRIAL DISCHARGES (30 DAY AVERAGE)

Industry	BOD	TSS	pH
Dairy - Fluid, Cultured Products	0.37 lb/100 lb. BOD[1]	0.046 lb/100 lb. BOD	6.0-9.0
Dairy - Butter, Cheese Processing	0.008 lb/100 lb. BOD	0.01 lb/100 lb. BOD	6.0-9.0
Dairy - Ice Cream Processing	0.47 lb/100 lb. BOD	0.059 lb/100 lb. BOD	6.0-9.0
Wet Corn Milling	20 lb/1000 bu. corn[2]	10 lb/1000 bu. corn	6.0-9.0
Parboiled Rice Processing	0.007 lb/100 wt. rice	0.003 lb/100 wt. rice	6.0-9.0
Tuna Processing	8.1 lb/1000 lb. tuna	3.0 lb/1000 lb tuna	6.0-9.0
Cane Sugar Refining	0.18 lb/ton of melt	0.07 lb/ton of melt	6.0-9.0
Wool Finishing (Textile)	11.2 lb/1000 lb. wool	11.2 lb/1000 lb. wool	6.0-9.0
Woven Fabric Finishing	3.3 lb/1000 lb. fabric	3.3 lb/1000 lb. fabric	6.0-9.0
Carpet mill	3.9 lb/1000 lb	3.9 lb/1000 lb.	6.0-9.0
Cement manufacturing	--	0.005 lb/1000 lb. product	6.0-9.0
Electro plating	--	327 lb/10^6 ft^2	6.0-9.5
Organic Chemicals (aqueous)	0.2 lb/1000 lb.	0.12 lb/1000 lb.	6.0-9.0
Plastics (Polyvinyl Chloride) Manufacture	0.19 lb/1000 lb.	0.13 lb/1000 lb.	6.0-9.0
Rayon manufacture	2.0 lb/1000 lb.	1.28 lb/1000 lb.	6.0-9.0
Acrylics manufacture	0.87 lb/1000 lb.	0.27 lb/1000 lb.	6.0-9.0
Soap Production (by Batch Kettle)	0.40 lb/1000 lb.	0.40 lb/1000 lb.	6.0-9.0
Petroleum cracking	3.1 lb/1000 bbl. feedstock[3]	2.0 lb/1000 bbl. feedstock[3]	6.0-9.0
Petrolchemical processing	4.1 lb/1000 bbl. feedstock[3]	2.7 lb/1000 bbl. feedstock[3]	6.0-9.0
Iron & Steel (Coke By Product)	--	0.104 lb/100 lb.	6.0-9.0
Iron (Blast Furnace)	--	0.13 lb/1000 lb.	6.0-9.0
Primary Alumnium Smelting	--	0.05 lb/1000 lb. product	6.0-9.0
Hide Tanning and Finishing	1.6/1000 lb.	2.0 lb/1000 lb.	6.0-9.0

TABLE III-38 (continued)

Industry	BOD	TSS	pH
Asbestos - Cement Pipe	--	0.38 lb/1000 lb.	6.0-9.0
Rubber Tire Production	--	0.064 lb/1000 lb.	6.0-9.0
Kraft Paper Process	3.1 lb/ton	7.5 lb/ton	6.0-9.0
Slaughterhouse Operation	0.12 lb/1000 live weight	0.2 lb/1000 lb. live weight	6.0-9.0

[1] Units are in lbs of effluent BOD per 100 lbs of influent BOD.

[2] Reference to weight of products are to weights of raw input product.

[3] Actual limits equal listed limits times facility size dependent multipliers

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CHAPTER 4

RIVERS AND STREAMS

4.1 INTRODUCTION

The main purpose of the formulations presented in this chapter is to allow the user to predict responses of rivers and streams to selected waste loading schemes. It is suggested that the planner read this introductory section completely to gain a feel for the scope of problems that will be covered and the limitations of formulations presented.

Rivers throughout this country are subject to a wide spectrum of geological, biological, climatological, and man-induced impacts, and the result is a variety of water quality problems. Any approach providing guidance to the solution of these problems, especially one restricted to hand calculations, must be limited in scope. The following guidelines have been used in selecting topics to be considered within this section: 1) widely occurring problems, 2) those amenable to hand calculations, and 3) those for which planners will be able to obtain sufficient data.

4.1.1 Scope

The major problem areas to be considered are:

- Carbonaceous (CBOD) and Nitrogenous (NBOD) Biochemical Oxygen Demand
- Dissolved Oxygen
- Temperature (with a discussion of low flow)
- Nutrients and Eutrophication Potential
- Total Coliforms
- Conservative Constituents
- Sedimentation and Suspended Solids

Since 1974, the U.S. Environmental Protection Agency has annually published the National Water Quality Inventory which is a compilation of current water quality conditions and recent trends in the nation's rivers and lakes. Several of the tables in that report series are relevant to this document and are included here. Table IV-1 illustrates reference water quality levels used to define acceptable pollutant limits in U.S. waterways. Table IV-2 shows water quality conditions in eight major waterways in the United States, while Table IV-3 summarizes the most widely observed water quality problems in the U.S. These tables will frequently be cited throughout this chapter.

4.1.2 Significance of Problem Areas

Oxygen depletion is often the result of excessive CBOD and NBOD loadings particularly in combination with high temperature, low flow conditions. Increased nutrient loadings to streams producing elevated ambient concentrations pose substantial potential for eutrophication. The nutrient problem is currently one of the most widespread areas of concern regarding river water quality. The health hazards category in Table IV-3 lists elevated coliform levels, a problem of particular concern in northeastern and Great Lakes States. Salinity also has been identified as a major problem, particularly in the central and southwestern states.

Because of their importance, each of the problem areas just described will be addressed in this chapter. As shown in Table IV-4, many states routinely measure the parameters associated with these problems. The total number of states responding to the survey was 47. Because of the routine surveys conducted, data are commonly available for performing hand calculations. NBOD, though not directly measured, can be found from measurements of organic and ammonia nitrogen. Chloride concentration measurements can be directly converted to salinity.

TABLE IV-1

REFERENCE LEVEL VALUES OF WATER QUALITY INDICATORS FOR U.S. WATERWAYS (EPA,1974)

Parameter	Reference Level, Receptor, and Information Source
Suspended solids	80 mg/l - aquatic life+
Turbidity	50JTU - aquatic life*
Temperature	90°F - aquatic life*
Color	75 platinum-cobalt units - water supply+
Ammonia †	0.89 mg/l - aquatic life+
Nitrate (as N)	0.9 mg/l - eutrophication
Nitrite plus nitrate †	0.9 mg/l - eutrophication
Total phosphorus †	0.1 mg/l - eutrophication
Total phosphate †	0.3 mg/l - eutrophication
Dissolved phosphate †	0.3 mg/l - eutrophication
Dissolved solids (105°C)	500 mg/l - water supply*
Dissolved solids (180°C)	500 mg/l - water supply*
Chloride	250 mg/l - water supply+
Sulfate	250 mg/l - water supply+
pH	6.0-9.0 - aquatic life*
Dissolved oxygen	4.0 mg/l - aquatic life*
Total coliforms (MFD)**	10,000/100 ml - recreation*
Total coliforms (MFI)**	10,000/100 ml - recreation*
Total coliforms (MPN)**	10,000/100 ml - recreation*
Fecal coliforms (MF)**	2,000/100 ml - recreation*
Fecal coliforms (MPN)**	2,000/100 ml - recreation*
Phenols	0.001 mg/l - water supply+

*Guidelines for Developing or Revising Water Quality Standards, EPA Water Planning Division, April 1973.

+Criteria for Water Quality, EPA, 1973 (Section 304(a)(1) guidelines).

††Biological Associated Problems in Freshwater Environments, FWPCA, 1966, pp 132-3.

**Membrane filter delayed, membrane filter immediate, most probable number, membrane filter.

† Original reference did not specify whether as N or NO_3, or P or PO_4.

TABLE IV-2

CONDITION OF EIGHT MAJOR WATERWAYS (EPA,1974)

River	Harmful Substances	Physical Modification	Eutrophication Potential
Mississippi		High* turbidity and solids below Missouri River	High* increasing nutrients but no algae
Missouri	Trace metals present in middle river	High* suspended solids, turbidity in middle and lower river	High*, increasing nutrients but no algae
Ohio	High*; increasing iron and manganese	High* suspended solids in lower river, some improvements	High* nutrients but no algae
Tennessee			Small increase in nutrients but no algae
Detroit area rivers	Cyanide present but improving	Suspended solids improving, local temperature effects from discharges	Nutrients discharged to Lake Erie decreasing
Columbia	Severe gas super saturation; some radioactivity in lower river	Occasional high* temperatures	High* nutrients but no algae, except for slime growths in lower river
Snake	Severe gas supersaturation, significant pesticides	Turbidity from natural erosion, agricultural practices, reservoir flushing	Nuisance algal blooms each summer

TABLE IV-2 (continued)

River	Harmful Substances	Physical Modification	Eutrophication Potential
Willamette	Significant sulfite waste liquor from pulp and paper wastes	High* turbidity at high flow, high temperature in summer	High* level of nutrients but not excessive algae

River	Salinity, Acidity, and Alkalinity	Oxygen Depletion	Health Hazards and Aesthetic Degradation
Mississippi	High* salinity, acidity below major tributaries	Oxygen-demanding loads from large cities evident	Commercial fishing eliminated in lower river by phenols, bacteria near cities
Missouri	High* dissolved salts in middle and lower river	High* organic loads from feedlots, improved near cities	High* bacteria and viruses present in wet and dry periods
Ohio	Low* alkalinity especially in upper river	Occasional low* dissolved oxygen near Cincinnati and Pittsburgh	High* bacteria especially in high population areas
Tennessee		Low* BOD and decreasing COD in reservoirs	High* bacteria in small areas near cities, low radionuclides
Detroit area rivers	Acids and chloride low,* improving despite large discharges	Low* dissolved oxygen only at mouths of area tributaries	Phenols decreasing, bacteria unchanged-to-higher

TABLE IV-2 (continued)

River	Salinity, Acidity, and Alkalinity	Oxygen Depletion	Health Hazards and Aesthetic Degradation
Columbia	Approaches ideal for fresh waters	Dissolved oxygen close to saturation	Very low* bacteria
Snake	High* dissolved solids from irrigation in middle river	Dissolved oxygen close to saturation	High* bacteria below population centers
Willamette	Low* dissolved mineral salts, improved pH	Improved dissolved oxygen, no standards violations	High* bacteria, but improving

---

*High (or low) relative to other rivers, or relative to other sections of river, or to national reference levels. Does not necessarily imply standards violations or dangerous condition.

TABLE IV-3

WATER QUALITY PROBLEM AREAS REPORTED BY STATES* NUMBER REPORTING PROBLEMS/TOTAL (EPA,1975)

	Middle Atlantic, Northeast	South	Great Lakes	Central	Southwest	West	Islands	Total
Oxygen depletion	11/13	9/9	6/6	6/8	4/4	6/6	4/6	46/52
Eutrophication potential	11/13	6/9	6/6	8/8	2/4	6/6	4/6	43/52
Health hazards	11/13	8/9	5/6	8/8	3/4	5/6	5/6	45/52
Salinity, acidity, alkalinity	3/13	6/9	2/6	6/8	4/4	4/6	2/6	27/52
Physical modification	7/13	3/9	3/6	8/8	3/4	6/6	5/6	35/52
Harmful substances	6/13	6/9	5/6	4/8	4/4	2/6	3/6	30/52

* Localized or statewide problems discussed by the States in their reports.

TABLE IV-4

WATER QUALITY PARAMETERS COMMONLY MONITORED BY STATES* (EPA,1975)

Parameter	Number of States
Flow	47
Dissolved oxygen	47
Coliform bacteria	45
Nitrogen (any form)	39
Phosphorus (any form)	35
pH	35
BOD/COD/TOC	27
Water temperature	29
Turbidity	26
Solids (any type)	27
Metals (any type)	17
Chlorides	19
Alkalinity	15
Conductivity	16
Color	11
Sulfate	14

*Only parameters listed by at least 10 States and specified as being part of each State's monitoring program are included.

4.1.3 Applicability to Other Problems

The basic theories to be presented are aimed at reaching solutions to particular water quality problems. Generally, however the theories have broader applicability than will be presented, in terms of additional water quality parameter analysis. The approach most often utilized below is the law of mass conservation. In the case of temperature, an energy balance is performed which is analogous to a mass balance. The degree of commonality of source and sinks of a particular pollutant (e.g., nutrients) or water quality indicator (e.g., dissolved oxygen) is responsible for the similarities and differences among the specific equations. For example, CBOD and NBOD produce a similar general effect (oxygen depletion), generally have similar sources and sinks, and for purposes of this study, are assumed to follow first order decay kinetics. Coliforms, also assumed to decay by first order kinetics, are handled by the mass-balance approach. Conservative substances are different from BOD and coliforms in that they do not decay. Finally, there are also some instances where a more subjective analysis is indicated, and neither a mass nor energy balance is presented.

Once the similarities among water quality parameters are understood, handling two seemingly different problems may be accomplished in a straightforward fashion. More generally, it is possible to apply the tools presented here to other problems which, although not specifically discussed, can be analyzed using the same basic tools. For example, the distribution of toxic substances that are either conservative or follow a first order decay may be evaluated using techniques described for conservative substances and coliforms, respectively.

4.1.4 Sources of Pollutants

Pollutant loadings can be classified as originating from three general sources: point, nonpoint, and natural. Each of these can constitute a major hurdle in meeting the 1983 goals of fishable and swimmable waters. Specifically, point sources (30 states), nonpoint sources (37 states), and natural conditions (21 states) are all major contributors to water quality problems (EPA, 1975).

It is imperative that the capacity to accommodate nonpoint sources be a part of the hand calculation tool for rivers. Table IV-5 illustrates, for example, the importance of nonpoint source nutrient loading for selected rivers in Iowa. Up to 95.9% of the annual phosphorus load and up to 99.1% of the total nitrogen are from nonpoint sources. Admittedly, quantification of these loads is often difficult. Nevertheless, simplified nonpoint source terms will be included in some of the mass-balance formulations since Chapter III has supplied a methodology for estimating nonpoint source loading rates.

4.1.5 Assumptions

In deriving the mass-balance equations, it is necessary to make certain assumptions. The planner should have an understanding of these in order to prevent the misapplication of developed formulations. The major assumptions are:

- steady-state
- plug flow
- vertically and laterally mixed system
- first order decay rates (when decay occurs)

The steady-state assumption means that conditions are not changing with time. Changes are considered only as a function of distance along the river, and the time scale to be considered for steady-state generally should be on the order of a week or longer. For example, summer low flow conditions can be assumed to represent a steady-state. However, storm events, and the dynamic responses of a river to them, must be considered a transient situation.

TABLE IV-5

ANNUAL PHOSPHORUS AND NITROGEN LOAD FOR SELECTED IOWA RIVER BASINS (EPA,1975)

ANNUAL PHOSPHORUS LOAD

River	Total (lbs/year)	Point Sources (lbs/year)	Nonpoint Sources (lbs/year)	Percent of Total from Nonpoint Sources
Floyd	720,207	29,807	690,400	95.9
Little Sioux	1,851,632	129,088	1,722,544	93.0
Chariton	879,916	48,203	831,713	94.5
Des Moines	5,621,007	586,015	5,034,992	89.6
Iowa	1,723,975*	103,445*	1,620,530*	94.0
Cedar	5,099,507	1,526,775	3,572,732	70.1

*Orthophosphate

ANNUAL NITROGEN LOAD

River	Total (lbs/year)	Point Sources (lbs/year)	Nonpoint Sources (lbs/year)	Percent of Total from Nonpoint Sources
Floyd	1,705,984	65,171	1,640,813	96.2
Little Sioux	9,609,556	85,308	9,522,248	99.1
Chariton	1,585,427	24,795	1,560,632	98.4
Des Moines	41,334,897	695,235	40,639,662	98.3
Iowa	2,075,830	91,287	1,984,543	95.6
Cedar	6,804,881	1,552,334	5,252,547	77.2

Plug flow assumes that the effects of dispersion can be neglected. If an instantaneous slug of dye were injected into a stream with purely plug flow, there would be no tendency for the plug to spread out due either to a concentration gradient between the dye and the surrounding water or to velocity gradients. In actuality, some mixing does occur. Since the effect of dispersion is to spread pollutants, concentrations predicted along a river may be higher under the plug flow assumption than would actually be observed.

The fully-mixed assumption presupposes a well mixed system. That is, concentration gradients exist only in the direction of flow (longitudinal direction) and not in either the vertical or lateral direction. The final major assumption is that all decay rates can be approximated by first order decay. This means that the decay rate of a substance is proportional to the amount present. First order decay is traditionally used in CBOD computations, and occasionally in nitrogen oxidation. The oxidation of inorganic nitrogen actually proceeds in stages from ammonia-N to nitrite-N to nitrate-N. However, for purposes of this report, the first order decay rate is acceptable for NBOD and coliforms, as well as CBOD. Before applying first order decay to other substances, however, care should be taken to determine the validity of this assumption.

4.1.6 Spatial Variation

For the analysis of spatial variation, the portion of the river under investigation is divided into reaches. Generally within each reach a fairly constant stream temperature, velocity, depth, and cross-sectional area must be assumed. In the discussions below, some exceptions will occasionally be made. These are noted wherever they occur.

Any incoming point source or major tributary should constitute the beginning of a new reach. If a distributed source is included, its influence is assumed to extend over the entire reach. The more complicated the situation the planner wants to simulate, the larger the number of reaches into which a given portion of river will have to be divided.

4.1.7 Data Requirements

Associated with any of the water quality problems to be considered is the compilation of a small amount of data. For example, stream velocity (U), volumetric flow rate (Q), and stream temperatures (T) are needed in the analysis of many water quality problems. Specific to each problem are numerous other parameters, such as decay rates and incoming biochemical oxygen demand, for which estimates are needed.

When it is necessary to gather data, such as stream velocity (U_0) at the upstream end of a reach, caution and judgment should always be exercised. U_0, for example, should be measured downstream from any incoming waste flow or tributary entering at the upper end of the reach. The location of measurements should be representative of the river configuration in the vicinity of the upstream end of the reach. The same judgment should apply to measurements of stream cross-sectional areas where needed. In such cases, representative areas should be chosen. By attempting to understand the significance of each parameter utilized, the user is more likely to arrive at a meaningful and realistic solution than if he proceeds mechanically.

4.1.8 Selection of Season

It is reasonable to expect that a particular water quality problem may be more severe at one time of the year than another. Table IV-6 shows that pollutant levels can depend on season (summer or winter) and flow rate (high flow or low flow). Dissolved oxygen problems, for example, are clearly associated with summer, low flow conditions. Consequently, for any particular pollution problem, the planner should strive

TABLE IV-6

MAJOR WATERWAYS: SEASONAL AND FLOW ANALYSIS, 1968-72 (EPA, 1974)

Parameters	Winter, High Flow	Summer, Low Flow	Winter, Low Flow	Summer, High Flow	Dominant Effect
	(number of reaches exceeding reference levels)**				
Suspended solids	9	5	0	4	High flow
Turbidity	13	4	1	7	High flow
Color	11	6	3	4	High flow
Ammonia	14	3	7	1	Cold weather
Nitrite	3	7	5	1	Low flow
Nitrate(as N)	12	4	8	1	Cold weather
Nitrate(as NO_3)	8	3	6	1	Cold weather
Nitrite plus nitrate	2	3	2	1	Inconclusive
Organic nitrogen	3	6	0	3	Warm weather
Total Kjeldahl nitrogen	3	5	0	3	Warm weather
Total phosphorus	10	3	5	2	Cold weather
Total phosphate	8	3	5	1	Cold weather
Dissolved phosphate	6	3	4	0	Cold weather
Dissolved solids(105°C)	4	7	6	3	Low flow
Dissolved solids(180°C)	3	8	6	2	Low flow
Chlorides	4	15	10	0	Low flow
Sulfates	5	13	5	5	Warm weather,low flow
Alkalinity	6	12	10	0	Low flow
ph	15	4	6	4	Cold weather,high flow
Dissolved oxygen	0	19	0	9	Warm weather
BOD_5	11	6	8	1	Cold weather
COD (.025N)	6	5	3	2	Cold weather
Total coliforms(MFD)*	4	10	2	5	Warm weather
Total coliforms(MFI)*	8	6	2	4	High flow,warm weather
Total coliforms(MPN)*	4	2	3	3	Inconclusive
Fecal coliforms(MF)*	6	6	3	4	Inconclusive
Fecal coliforms(MPN)*	4	0	1	0	Cold weather
Phenols	5	0	1	0	Inconclusive
Odor	4	0	0	0	Inconclusive

*Membrane filter delayed, membrane filter immediate. most probable number, membrane filter.

**Reference levels are available in Table IV-1. Thirty reaches were analyzed during each season.

to perform the analysis under critical conditions normally encountered. Where planning is performed with consideration of the aggrevated situation, and where proper abatement action is taken, it is likely that pollution concentrations will be below problem levels during other times of the year. If a problem in fact exists, then it is under these conditions that it will be most pronounced.

In the following sections, hand calculation methods for each problem area are described with illustrative examples. Table IV-7 provides a cross-reference of the material presented.

4.2 CARBONACEOUS AND NITROGENOUS OXYGEN DEMAND

4.2.1 Introduction

Many wastes discharged into waterways contain biologically oxidizable materials that exert an oxygen demand on waterway resources. This biochemical oxygen demand (BOD) can be subdivided into carbonaceous (CBOD) and nitrogenous (NBOD) components. Table IV-8 illustrates typical concentrations of NBOD and CBOD in untreated municipal waste. The NBOD can be estimated by:

$$\text{NBOD} = 4.57\ (\text{TON})$$

where TON represents total oxidizable nitrogen. TON can be estimated by the sum of organic plus ammonia nitrogen. For example, a typical TON from Table IV-8 is 20 + 28 = 48 mg-N/l. This represents an NBOD of 219 mg/l, the same as the approximate average in that table.

TABLE IV-7

INTERRELATIONSHIPS BETWEEN SECTIONS OF CHAPTER 4

Section	Prerequisite Section	Related Sections	Relationship	Illustrative Examples
BOD	Introduction	Coliforms	First order decay	IV-1
DO	BOD, Introduction	Temperature	Critical DO levels	IV-2 → IV-5
Temperature	Introduction	DO	Temperature dependence	IV-6 → IV-8
Nutrients	Introduction	Conservative constituents	Nutrients considered conservative	IV-9
Total Coliforms	Introduction	BOD	First order decay	IV-10
Conservative Constituents	Introduction	Nutrients	Nutrients considered conservative	IV-11
Sedimentation	Introduction	Conservative constituents	Suspended solids conservative over long term	IV-12

TABLE IV-8

MUNICIPAL WASTE CHARACTERISTICS BEFORE TREATMENT (THOMANN, 1972)

Variable	Unit	Approx. Average	Normal Range
Average Daily Flow	gal/cap/day	125	100-200
Solids			
Total	mg/l	800	450-1200
Total Volatile	mg/l	400	250-800
Total Dissolved	mg/l	500	300-800
Total Suspended	mg/l	300	100-400
Volatile Suspended	mg/l	130	80-200
Settleable	mg/l	150	-
BOD			
Carbonaceous (5 day)	mg/l	180	100-450
Carbonaceous (ultimate)	mg/l	220	120-580
Nitrogenous*	mg/l	220	-
Nitrogen			
Total	mg/l N	50	15-100
Organic	mg/l N	20	5-35
Ammonia	mg/l N	28	10-60
Nitrite + Nitrate	mg/l N	2	0-6
Phosphate			
Total	mg/l PO_4	20	10-50
Ortho	mg/l PO_4	10	5-25
Poly	mg/l PO_4	10	5-25
Coliforms			
Total	million org./100 ml	30	2-50
Fecal	million org./100 ml	4	0.3-17

*Ultimate, Nitrogenous oxygen demand, exclusive of CBOD.

CBOD is a commonly measured characteristic of waste water. The CBOD used in the formulations presented below is the ultimate CBOD, the oxygen required for complete stabilization of the waste. Often CBOD is expressed as $CBOD_5$, the oxygen utilized in a 5 day test. The relationship between ultimate ($CBOD_L$) and 5-day CBOD can be approximated by:

$$CBOD_L = \frac{CBOD_5}{0.68}$$

The mass balance equation used in the CBOD analysis is exactly analogous to the NBOD equation. The first order decay rate assumption for NBOD stabilization is necessary to maintain this analogy, and is sufficient for hand calculations.

Nitrification (the process by which ammonia is oxidized to nitrite, and nitrite to nitrate) is pH dependent with an optimum range of 8.0 to 8.5 (Wild, 1971). To be more conservative, however, if the pH is between 7.0 and 8.5, significant nitrification may be assumed. If the pH is much lower than 7.0, nitrification is not likely to be important.

4.2.2 BOD Decay Rate

The decay rate for CBOD will be denoted by k_L and for NBOD by k_N. Typical values of both k_L and k_N lie between 0.1 and 0.6/day, with 0.3/day being typical. k_L values can, however, exceed the range given here. Values of 1 to 3/day have been computed for shallow streams (Thomann, 1972). A figure to be presented shortly will show how k_L depends on depth. The following discussion will be directed toward k_L, but in general will also apply to k_N.

The disappearance of BOD from a river is a reflection of both settling and biochemical oxidation, as shown in Figure IV-1. Biochemical oxidation can consist of instream oxidation (k_1L) as well as absorption by attached organisms (k_4L). The total oxidation rate then, is k_d, where

$$k_d = k_1 + k_4$$

The total loss rate k_L is

$$k_L = k_d + k_3$$

where k_3 reflects settling.

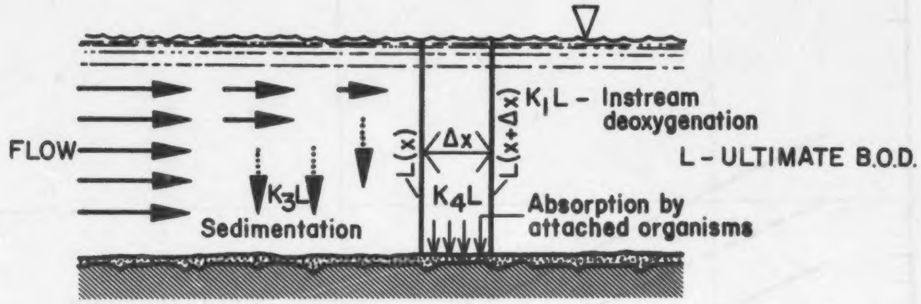

FLOW
K3L
Sedimentation
L(x)
Δx
L(x+Δx)
K1L - Instream deoxygenation
L - ULTIMATE B.O.D.
K4L
Absorption by attached organisms

FIGURE IV-1 MECHANISMS OF BOD REMOVAL FROM RIVERS

Settling of BOD is generally more prevalent just below a sewage discharge where the discharged material may contain a large suspended fraction. As this material is transported downstream the settling component becomes less important and the reaction rate k_L approaches the oxidation rate k_d. In this chapter, the settling component will not be explicitly considered. Neglecting settling will tend to cause instream BOD levels to be somewhat higher than they actually might be along certain portions of a river. It should be noted that if instream BOD data are used to determine k_L (one such method will be explained in Figure IV-3) then the effect of settling is automatically included in k_L. A further discussion of benthic demand (as associated with dissolved oxygen utilization) can be found in Section 4.3.10.

Figure IV-2 illustrates the dependence of k_L on river depth. The highest deoxygenation rates occur in shallow streams with stable, rocky beds, reflecting the significance of attached biological organisms. Appendix C contains observed and predicted values of k_L for various natural streams.

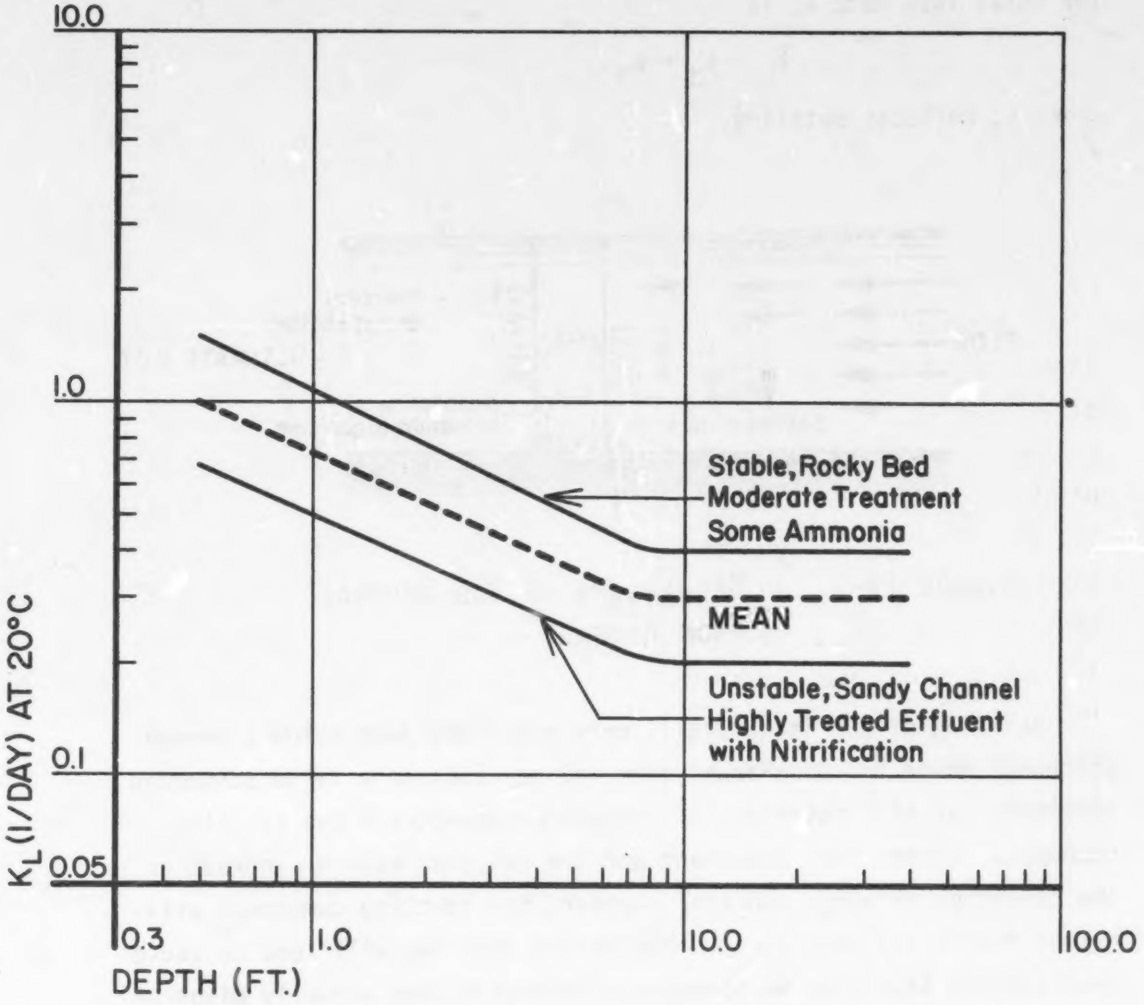


FIGURE IV-2 DEOXYGENATION COEFFICIENT AS A FUNCTION OF DEPTH, (AFTER HYDRO-SCIENCE, 1971)

The decay coefficients k_L and k_N are both temperature dependent and this dependence can be estimated by:

$$k_T = k_{20}\ 1.047^{(T-20)} \qquad \text{(IV-1)}$$

where

$k_{20} = k_L$ at $20^{o}C$

$k_T = k_L$ at $T^{o}C$

T = water temperature, ^{o}C

Numerous methods for computing k_L from observed data are available (Nemerow, 1974). One method entails the use of a semi-log plot. The stretch of the river containing the data to be plotted must have a constant stream area and flow rate, and the BOD loading must be from a point source located at a position that will be called $x = 0$. Plotting the log of BOD concentration versus distance generally produces a straight line with slope of $-k_L/U$. An example is shown in Figure IV-3. Either $CBOD_5$ or $CBOD_L$ can be plotted as the ordinate. Caution must be exercised to convert the slope from base 10 logarithms as given in the semi-log plot to base e logarithms as needed in the formulations used in this chapter. The conversion may be made by multiplying the value for log base 10 by 2.303.

4.2.3 Mass Balance of BOD

The general mass-balance equation for BOD with the first order decay process is

$$\frac{\partial L}{\partial t} = 0 = \frac{-1}{A} \frac{\partial}{\partial x} (QL) - k_L L + L_r \left(\frac{\partial Q}{\partial x}\right)/A + L_{rd} \qquad \text{(IV-2)}$$

where

L = CBOD (ultimate) remaining to be oxidized, mg/l
Q = volumetric flow rate, ft^3/sec
A = cross-sectional area, ft^2

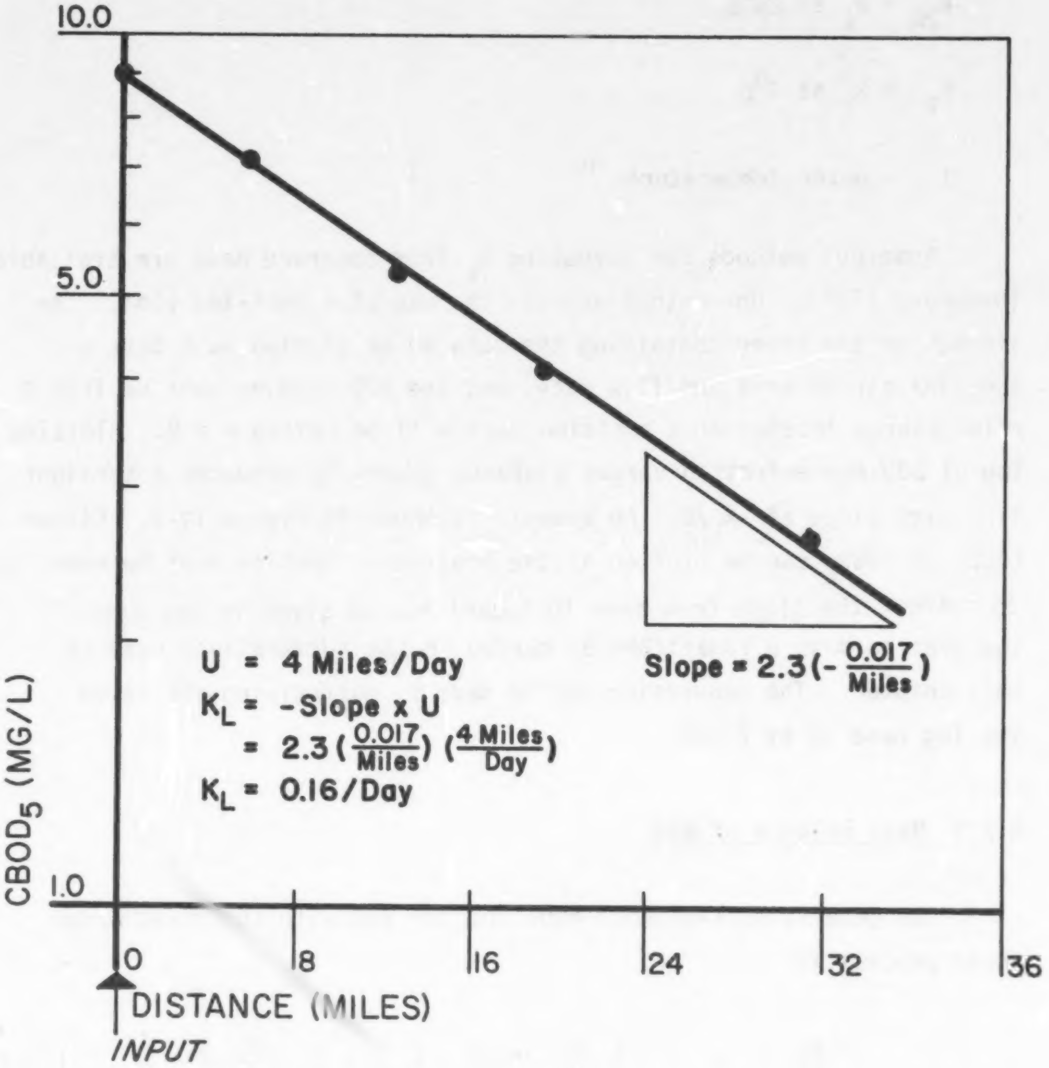


FIGURE IV-3 EXAMPLE OF COMPUTATION OF K_L FROM STREAM DATA (FROM HYDROSCIENCE, 1971)

L_r = concentration of CBOD entering through an incremental sideflow (distributed source), mg/l

L_{rd} = mass flux of CBOD entering, with no associated flow, mg/l/sec

x = stream distance

$\frac{\partial L}{\partial t}$ = 0 indicates that steady-state conditions are being assumed and thus no accumulation of material takes place at any point within the reach.

The NBOD equation is completely analogous in form to Equation IV-2:

$$\frac{\partial N}{\partial t} = 0 = -\frac{1}{A}\frac{\partial}{\partial x}(QN) - k_N N + N_r\left(\frac{\partial Q}{\partial x}\right)/A + N_{rd} \qquad \text{(IV-3)}$$

where N is the NBOD.

N_{rd} represents purely a mass flux of nitrogenous material, while $N_r\left(\frac{\partial Q}{\partial x}\right)/A$ is a source of NBOD entering the river reach through an incremental sideflow. Thus, in cases where a known distributed source of BOD significantly contributes to a river reach under study, and the distributed flow (flow associated with a distributed source) can be neglected, N_{rd} can be used in lieu of $N_r\left(\frac{\partial Q}{\partial x}\right)/A$. N_{rd} can be estimated by determining the mass M of BOD entering a volume of river water V in time T. N_{rd} is given by:

$$N_{rd} = \frac{M}{VT}$$

For any particular reach of a river under investigation the stream cross-sectional area can be expressed by:

$$A = A_o + \left(\frac{A_f - A_o}{x_L}\right)x = A_o + \Delta_A x \qquad \text{(IV-4)}$$

where

$$\Delta_A = \frac{A_f - A_o}{x_L}$$

A_o = stream area at upstream end of the reach
A_f = stream area at downstream end of reach
x = distance downstream from beginning of reach
x_L = length of reach

The cross-sectional area need not be measured directly, but can be computed from:

$$A = \frac{Q}{U}$$

The cross-sectional area change is a reflection of a change in stream velocity, perhaps due to a bed slope increase or decrease. The length of the reach under investigation, x_L, is measured in river miles along the river's centerline. If use of a constant stream area is assumed, then $\Delta_A = 0$ and $A = A_0$ throughout the reach.

4.2.4 Typical Solutions

Case 1: The only source of CBOD occurs as a point source at $x = 0$. The CBOD distribution is then expressed by:

$$L = L_o \exp\left[\frac{-j_L}{A_o} \quad (A_o x + \Delta_A \frac{x^2}{2})\right] \qquad \text{(IV-5)}$$

where

$$j_L = \frac{-k_L}{U_o}$$

U_o = stream velocity at $x = 0$
L_o = ultimate BOD at the upstream end of the reach
L = ultimate BOD at a distance x downstream

The other terms have previously been defined. The initial CBOD, L_o, must reflect both CBOD upstream of the reach as well as that contributed by the point source in question. It is given by:

$$L_o = \frac{L_u Q_u + W}{Q_u + Q_W}$$

where

W = mass rate of discharge of CBOD, lb/day
Q_u = upstream river flow, cfs
Q_W = waste flow rate, cfs
L_u = upstream CBOD concentration, mg/l

Case 2: For a point source of CBOD at $x = 0$ and a distributed mass influx of CBOD entering the river throughout the reach, the solution is

$$L = \frac{L_{rd}}{k_L} + \left(L_o - \frac{L_{rd}}{k_L}\right) \exp\left[\frac{-j_L}{A_o}\left(A_o x + \Delta_A \frac{x^2}{2}\right)\right] \qquad \text{(IV-6)}$$

where

L_{rd} = mass rate of CBOD entering the reach per unit volume of river water, mg/l/day

Case 3: A distributed flow enters the river carrying CBOD and a point source of CBOD exists at $x = 0$. The flow rate Q at a distance x is:

$$Q = Q_o + \frac{Q_f - Q_o}{x_L} x = Q_o + \Delta_Q x$$

where

$$\Delta_Q = \frac{Q_f - Q_o}{x_L}$$

The BOD distribution is given by (the river cross-sectional area is assumed constant throughout the reach):

$$L = \frac{L_r}{E_1} + \left(L_o - \frac{L_r}{E_1}\right)\left(\frac{Q_o}{Q}\right)^{E_1} \qquad \text{(IV-7)}$$

where

$$E_1 = \frac{k_L A_o + \Delta_Q}{\Delta_Q}$$

L_r = concentration of CBOD entering the river in the distributed flow, mg/l

Case 3 can also be used to establish the effect a purely diluting inflow (i.e. $L_r = 0$) would have on the CBOD distribution.

Case 4: For a point source at x = 0, a distributed source with associated inflow, and a mass flux with no associated flow (constant river cross-sectional area), the solution is

$$L = L_o\left(\frac{Q_o}{Q}\right)^{E_1} + \frac{L_r\Delta_Q + L_{rd}A_o}{k_L A_o + \Delta_Q}\left(1 - \left(\frac{Q_o}{Q}\right)^{E_1}\right) \qquad \text{(IV-8)}$$

where

$$E_1 = \frac{k_L A_o + \Delta_Q}{\Delta_Q}$$, as in Case 3.

4.2.5 Other Simplifying Procedures

The formulations represented by Equations IV-5 through IV-8 offer a range of options with which to examine BOD distribution in rivers. However, there are additional methods of estimating instream concentrations and determining whether or not significant BOD levels exist.

Perhaps the simplest method is assuming that BOD does not decay. An upper limit of the instream concentration at any point can then be determined by incorporating all known sources, and using the methods presented in Section 4.7. If the computed instream concentrations are below a threshold pollution level, then there is no need to apply Equations IV-5 through IV-8 because the inclusion of a decay rate can only lower the concentrations.

It may also be feasible, as a first estimate, to combine the CBOD and NBOD equations into one, and use that equation to estimate the distribution of the total oxygen-demanding material. To do this, all source terms must include both CBOD and NBOD. One decay coefficient is used for both CBOD and NBOD decay. The larger decay coefficient of the two might be used since that will produce the larger oxygen deficit.

In deciding which of Equations IV-5 through IV-8 to use for any analysis, the purpose of the analysis as well as data availability should be considered. If the main purpose is to relate *differences* in stream concentrations caused by various levels of abatement at a sewage treatment plant, the diffuse sources of BOD need not be considered. The resulting concentration difference can be expressed as:

$$\Delta L = \begin{cases} \Delta L_o \exp\left[\frac{-j_L}{A_o}\left(A_o x + \Delta_A \frac{x^2}{2}\right)\right] & \text{(IV-9A)} \\ \Delta L_o \left(\frac{Q_o}{Q}\right)^{E_1} & \text{(IV-9B)} \end{cases}$$

where ΔL is the change in BOD concentration due to a change, ΔL_o, in the initial concentration. Equation IV-9A should be used for a Case 1 or Case 2 situation, and Equation IV-9B for Case 3 or Case 4. If an estimate of the absolute level of BOD is desired, however, then the appropriate expression containing the significant sources should be utilized. It should be noted that if the diffuse sources of BOD are large then the improvement of instream BOD concentrations by point source control will be relatively minor. In that case the planner should focus on nonpoint source control.

EXAMPLE IV-1

Estimating BOD Distribution in a River

Suppose the user wants to calculate the BOD distribution in the river shown below in Figure IV-4. There are nine point sources contributing

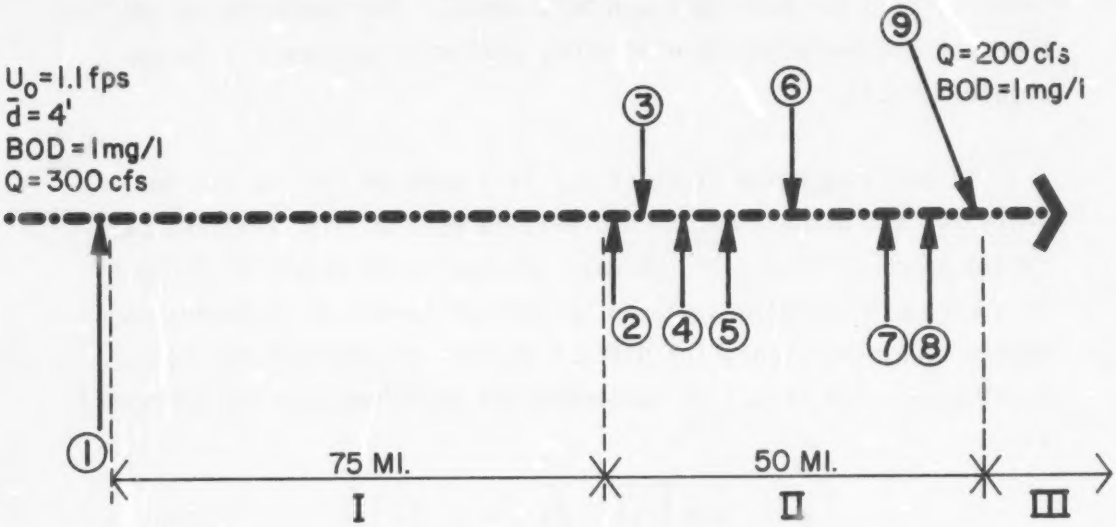


FIGURE IV-4 HYPOTHETICAL BOD WASTE LOADINGS IN A RIVER

BOD in the stretch of river under consideration. The ninth source is actually a tributary, and contributes substantially more flow than the other eight. The initial problem is to divide the river up into reaches. The first reach (I) should include the first 75 miles in which there is one point source of BOD at the upstream end (source (1)). Equation IV-5 is applicable to that reach. Now, there are several choices available regarding the division of the river between sources (2) and (8). One choice is to divide the 50 miles into mini-reaches similar to reach I, and reapply Equation IV-5 seven more times. A second alternative is to group adjacent point sources into fewer and larger sources, thereby requiring fewer applications of Equation IV-5. A third alternative is to assume that sources (2) through (8) comprise one continuous distributed source, the total pollutant loading of this equivalent source being equal to the sum of the individual loads. For this representation to be valid the sources should be both evenly spread spatially and be discharging comparable loads. The third alternative will be examined here, and reach II will consist of the 50 miles following reach I. Equation IV-7 will be used to analyze reach II. Reach III then, will begin just downstream from the tributary (source (9)).

For reach I, Equation IV-5 is first solved.* Suppose the following characteristics of waste source (1) are known:

$Q = 20 \text{ MGD} = 1.55\ (20) \text{ cfs} = 31 \text{ cfs}$

$W = 5000 \text{ lb. BOD}_5\text{/day}$

---

* Perhaps the most persistent problem encountered in performing an analysis similar to this is that of ensuring all units in the calculations are consistent. Two commonly used conversion factors in analyses of this type are:

$1 \text{ mg/l} = 8.34 \text{ lb/MG}$

$1 \text{ MGD} = 1.55 \text{ cfs}$

Recall that

$$L_o = \frac{L_u Q_u + W}{Q_u + Q_W}$$

W must be in lb. BOD ultimate/day:

$$W = \frac{5000}{.68} = 7353 \text{ lb. } BOD_L\text{/day}$$

Then

$$L_o = \frac{(1)\ (300) + 7353\ \frac{(1.55)}{8.34}}{300 + 31} = 5.0 \text{ mg/l}$$

The decay coefficient is estimated from Figure IV-2 as 0.4/day. No correction will be made for temperature. Equation IV-5 can now be expressed as (for constant cross-sectional area):

$$L = 5 \exp\left(\frac{-0.4}{(1.1)(24)(3600)}\ x\right)$$

where x is the downstream distance in feet. Note the correction needed to convert the decay coefficient from units of 1/day to 1/sec.

The results of the above equation for selected distances downstream can be expressed as follows:

x (miles)	L (mg/l)
0	5.0
30	2.6
60	1.3
75	0.9

For reach II, sources (2) through (8) are assumed to contribute the following loading.

$$\text{BOD} = 8000 \text{ lb/day}$$

$$Q = 120 \text{ MGD} = 186 \text{ cfs}$$

The flow distribution, Q, in reach II is then:

$$Q = Q_o + \frac{Q_f - Q_o}{x_L} x = 331 + \frac{186}{50} x$$

where x is in miles (from 0 to 50). L_r, the average BOD_L concentration in the incoming flow is:

$$L_r = \frac{8000 \text{ lb/day}}{120 \text{ MGD}} \times \frac{1 \text{ mg/l}}{8.34 \text{ lb/day}} = 8.0 \text{ mg/l}$$

If the average depth in reach II is assumed to be 5 feet, then:

$$K_L = .3/\text{day}$$

Finally, E_1 is computed:

$$E_1 = \frac{K_L A_o + \Delta_Q}{\Delta_Q}, \quad A_o = \frac{Q_o}{U_o} = \frac{331}{1.1} = 301 \text{ ft}^2$$

$$E_1 = \frac{\dfrac{(0.3)(301)}{(24)(3600)}}{\dfrac{186}{(50)(5280)}} + 1 = 2.5$$

Then, using L from the 75 mile point of Reach I as L_o:

$$L = \frac{8.0}{2.5} + \left(0.9 - \frac{8.0}{2.5}\right)\left(\frac{331}{Q}\right)^{2.5}$$

$$= 2.4 - 1.5\left(\frac{331}{Q}\right)^{2.5}$$

In tabulated form:

x (mi)	Q (cfs)	L (mg/l)
0	331	0.9
20	405	1.8
40	480	2.3
50	517	2.5

Note that the BOD concentration is increasing within this reach.

For reach III, only enough information is given to compute the initial concentration, which proportions the BOD at the end of reach II with that entering through the tributary (source (9)).

$$L_o = \frac{200(1) + 517(1.9)}{200 + 517} = 1.7 \text{ mg/l}$$

END OF EXAMPLE IV-1

4.2.6 Interpretation of Results

The most frequent use of BOD data in river water quality analyses involves their relationship with the dissolved oxygen balance. This relationship will be discussed more fully in Section 4.3. At this point it is sufficient to say that it is necessary to predict the BOD distribution in a river in order to compute dissolved oxygen concentrations.

When a river receives a heavy load of organic matter, the normal processes of self purification result in a series of zones of decreasingly severe conditions succeeding one another downstream. Each zone contains characteristic animals and plants (Nemerow, 1974). A saprobicity system (saprobicity is a measure of biodegradable organic matter) has been developed that relates BOD concentrations in streams to the degree of pollution there. Correlations have been found, for example, among BOD concentrations, coliform bacteria, and dissolved oxygen in rivers (Sladecek, 1965). Sladecek (1969) has assigned 5-day BOD values of 5 ppm to mildly polluted conditions and 10 ppm to substantial pollution.

Sources of drinking water are subject to restraints on the maximum allowable BOD that can be contained in raw water and still qualify as a drinking water source. Further, the degree of treatment of the raw water is dependent on the concentrations of certain constituents, such as BOD. One reference (HEC, 1975) has stated that water having a 5-day BOD over 4 mg/l, in combination with high levels of other constituents, represents a poor source of domestic water supply.

As discussed above, the concentration of BOD in a river can come from a number of sources, both point and nonpoint. It is important to have the capability of predicting what effect a reduction in waste loading will have on the instream concentration. One particular point source may, for example, contribute only a relatively small amount to the overall problem. Hence a BOD reduction from that particular point source, although it might be easiest to accomplish, might only produce negligible improvement.

4.3 DISSOLVED OXYGEN

4.3.1. Introduction

Historically dissolved oxygen has been and continues to be the single most frequently used indicator of water quality in streams and rivers. Figure IV-5 shows the seasonal variability of dissolved oxygen in 22 major waterways throughout the country (EPA, 1974) from 1968 to 1972. Invariably the levels observed from June to October are lower than those observed in January to March. This is due primarily to the influence of temperature on the dissolved oxygen levels. Due to the effect of temperature, summer is the most critical season in terms of pollutant assimilation in rivers.

The dissolved oxygen calculations presented below range in complexity from a simple CBOD-DO relationship to a more general dissolved oxygen mass balance including CBOD, NBOD, photosynthesis, respiration, and benthic demands. It should be stressed, however, that the results calculated from any of the relationships only provide estimates since each procedure incorporates various assumptions that are never fully met. For example, all natural rivers and streams exhibit varying degrees of dispersional effects, while the methods presented below do not include any dispersion. Also, waste loading inflows are assumed to remain constant in quality and quantity over time. In reality loadings may vary diurnally. Furthermore,

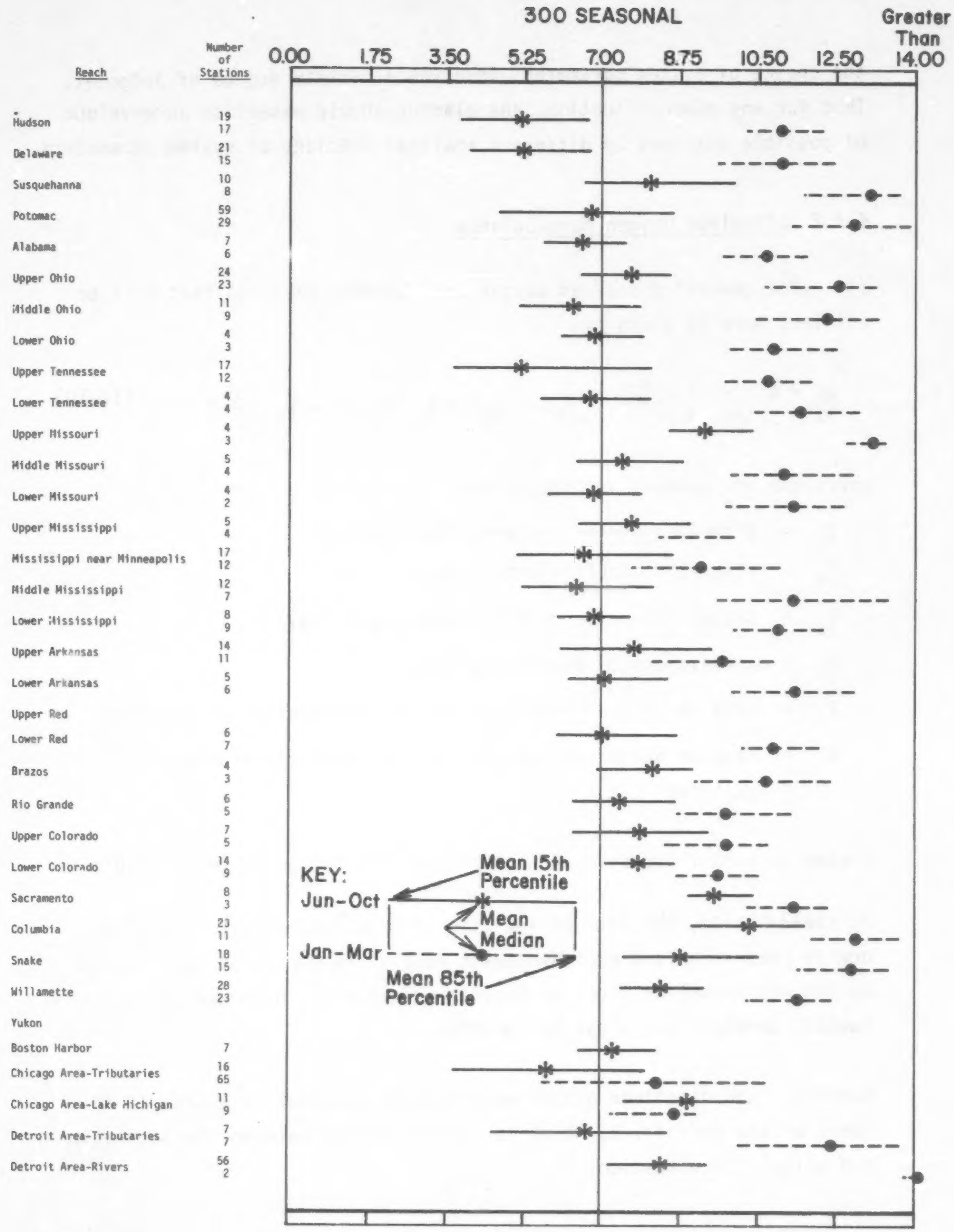


FIGURE IV-5 VARIABILITY OF DISSOLVED OXYGEN BY SEASON FOR 22 MAJOR WATERWAYS, 1968-72 (EPA, 1974)

the choice of system parameters involves a certain degree of judgment. Thus for any given situation, the planner should establish an envelope of possible outcomes by different realistic choices of system parameters.

4.3.2 Dissolved Oxygen Mass-Balance

The general dissolved oxygen mass-balance equation that will be utilized here is given by:

$$\frac{\partial C}{\partial t} = 0 = - \frac{1}{A}\frac{\partial(QC)}{\partial x} - k_L L - k_N N + k_a (C_s - C) - S_b + P - R \qquad \text{(IV-10)}$$

where the new symbols introduced are:

C = dissolved oxygen concentration, mg/l

k_a = reaeration coefficient, 1/day

C_s = saturation value of dissolved oxygen, mg/l

S_b = benthic oxygen demand, mg/l/day

P = rate of oxygen production due to photosynthesis, mg/l/day

R = rate of oxygen consumption due to algal respiration, mg/l/day

Stated in words, Equation IV-10 expresses the following relationship:

At steady state, the rate of addition of dissolved oxygen to a river due to reaeration and photosynthesis equals the depletion rate caused by the net advective flow, carbonaceous oxidation, nitrogenous oxidation, benthic demands, and algal respiration.

Commonly, the dissolved oxygen mass-balance equation is expressed in terms of the deficit, D, which is the difference between the saturation and actual concentrations.

4.3.3 Reaeration Rate

The atmosphere provides the major source for replenishing the dissolved oxygen resources of rivers. This mechanism tends to equilibrate the dissolved oxygen concentration in a river to its saturation value. Most commonly, the dissolved oxygen concentration is below saturation and there is a net influx of oxygen into the river from the atmosphere. On occasion, due to the production of dissolved oxygen by algae, rivers or streams can become supersaturated, in which case there is a net loss of oxygen to the atmosphere.

A number of expressions for the reaeration coefficient, k_a, have been developed. Two are presented here. O'Connor's formulation (Thomann, 1972) states that:

$$k_a = \frac{(D_L U)^{1/2}}{H^{3/2}} \text{ at } 20^{\circ}\text{C} \qquad \text{(IV-11)}$$

where

D_L = oxygen diffusivity = 0.000081 ft^2/hr at 20°C

H = stream depth in ft

U = stream velocity in ft/sec

Expressed in English units,

$$k_a = \frac{12.9\ U^{1/2}}{H^{3/2}} \text{ at } 20^{\circ}\text{C}$$

The above formula was verified on streams and rivers ranging in average depth from 1 foot to 30 feet with velocities ranging from 0.5 to 1.6 fps. Its use should be limited to streams where the reaeration coefficient is less than 12/day. Figure IV-6 illustrates how k_a changes with depth and velocity.

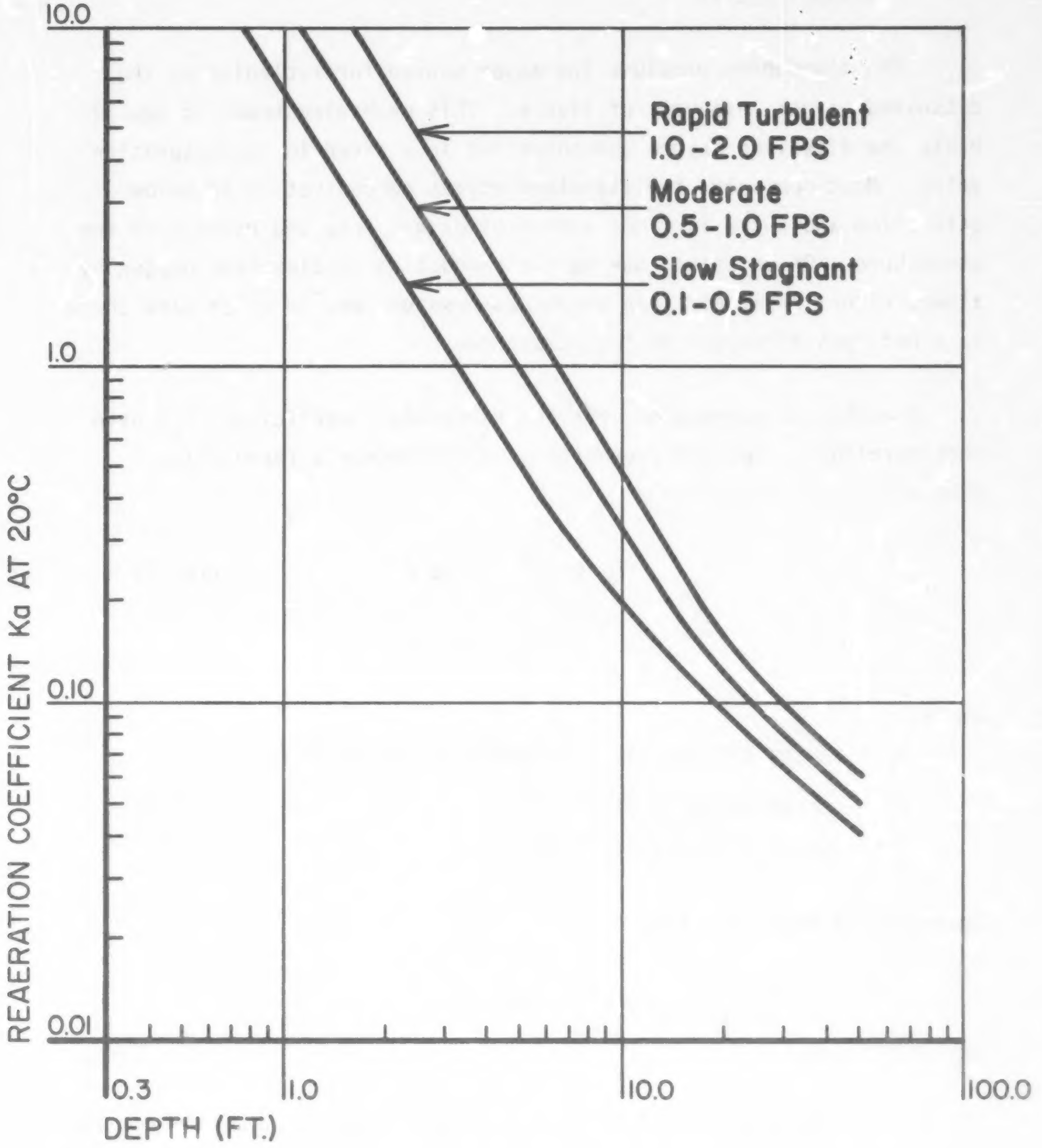


FIGURE IV-6 REAERATION COEFFICIENT AS A FUNCTION OF DEPTH (FROM HYDROSCIENCE, 1971)

For shallow (0.4 - 2.4 feet), fast moving streams the following expression developed by Owens (Thomann, 1972) is preferable, as the experimental work to develop this expression was done almost exclusively on shallow streams:

$$k_a = 21.6 \frac{U^{0.67}}{H^{1.85}} \text{ at } 20^oC \qquad \text{(IV-12)}$$

where U is in ft/sec and H in feet. A graphical representation of Equation IV-12 is shown in Figure IV-7.

Temperature changes affect the reaeration rate, and the relationship can be approximated by:

$$(k_a)_T = (k_a)_{20} \; 1.024^{(T-20)} \qquad \text{(IV-13)}$$

where $(k_a)_T$ is the reaeration coefficient at T ^oC.

In addition to temperature, substantial suspended sediment concentrations can appreciably alter the reaeration rate in streams (Alonso, et al, 1975). As an approximation, k_a decreases by 9% per 1,000 ppm increase in suspended sediment up to a 4,000 ppm load. Beyond that, concentration data are not available to assess the response of k_a. It is suggested that a 40% decrease be used for higher suspended sediment loads. Rivers with high suspended sediment loads are generally found in the western central states. Measured values of k_a for various streams and rivers are included in Appendix C.

4.3.4 Effect of Dams on Reaeration

Many rivers or streams have small to moderate sized dams crossing them in one or more places. Reaeration occurs as the water flows over the dam. Based on experimental data (Gameson, et al., 1958), and later verified with field data (Barrett, et al., 1960), the following relationship for reaeration over dams has been developed:

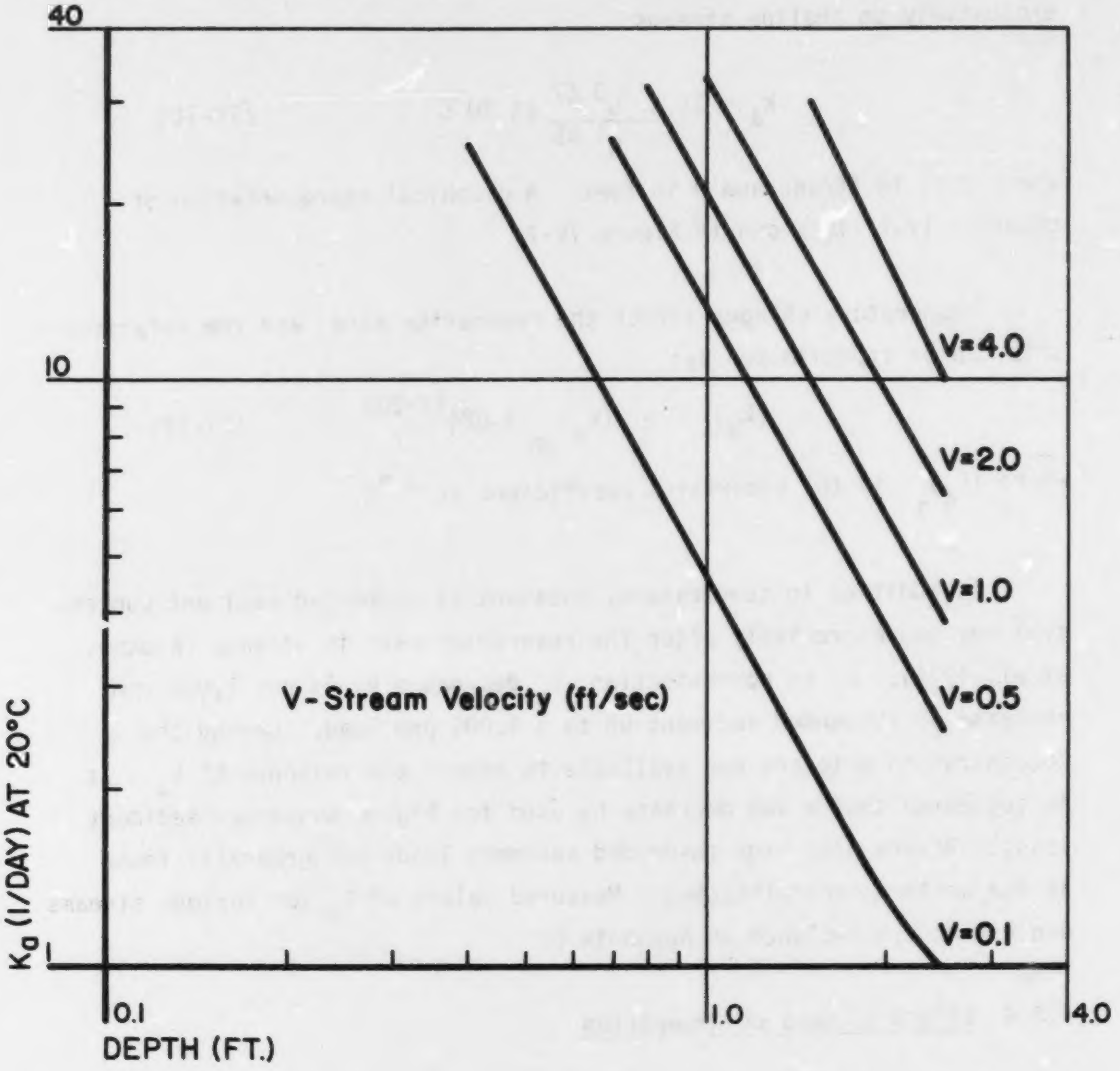


FIGURE IV-7 REAERATION COEFFICIENT FOR SHALLOW STREAMS, OWEN'S FORMULATION

$$D_a - D_b = \left[1 - \frac{1}{1 + 0.11\ ab(1 + 0.046T)H} \right] D_a \qquad \text{(IV-14)}$$

where

D_a = dissolved oxygen deficit above dam, mg/l

D_b = dissolved oxygen deficit below dam, mg/l

T = temperature, °C

H = height through which the water falls, ft

a = 1.25 in clear to slightly polluted water: 1.00 in polluted water

b = 1.00 for weir with free fall: 1.3 for step weirs or cascades

An alternate equation developed from data on the Mohawk River and Barge Canal in New York State (Mastropietro, 1968) is as follows:

$$D_a - D_b = 0.037H\ D_a \qquad \text{(IV-15)}$$

Equation IV-15 is valid for dams up to fifteen feet high and for temperatures in the range of 20° to $25^{\circ}C$. It is recommended that this equation be used within its stated range of applicability.

In handling the problem of a dam, a new reach can be started just below the dam. D_a can be calculated as the value that occurs at the end of the old reach. The new deficit D_b, which will become the deficit at the beginning of the next reach, is calculated from one of the above two formulas.

4.3.5 Dissolved Oxygen Saturation

The rate at which atmospheric reaeration occurs depends not only on k_a, but also on the difference between the saturation concentration C_s and the actual concentration C. The saturation value of dissolved oxygen is a function of temperature, salinity, and barometric pressure. The effect of salinity becomes important in estuarine systems, and to a lesser degree in rivers where high irrigation return flow can lead to substantial salinity values. Table IV-9 depicts the relationship between oxygen saturation and chlorinity. The expression relating salinity and chlorinity concentration is:

$$\text{Salinity } (^{o}/_{oo}) = 0.03 + 0.001805 \text{ chlorinity (mg/l)}$$

where $^{o}/oo$ represents parts per thousand.

The temperature dependence (at zero salinity) can be expressed as:

$$C_s = 14.65 - 0.41022T + 0.00791T^2 - 0.0007774T^3 \quad \text{(IV-16)}$$

where T is in °C. This relationship is also expressed in Table IV-8 for zero chloride concentration.

Barometric pressure affects C_s as follows:

$$C_s' = C_s \left(\frac{P_b - P_v}{760 - P_b} \right) \quad \text{(IV-17)}$$

where

C_s = saturation value at sea level, at the temperature of the water, mg/l

C_s' = corrected value at the altitude of the river, mg/l

P_b = barometric pressure at altitude, mm Hg

P_v = saturation vapor pressure of water at the river temperature, mm Hg

As an approximation of the influence of altitude, C_s decreases about 7% per 2,000 feet elevation increase.

TABLE IV-9

SOLUBILITY OF OXYGEN IN WATER (STANDARD METHODS, 1971)

Temp. in °C	Chloride Concentration in Water - mg/l					Difference per 100 mg Chloride
	0	5,000	10,000	15,000	20,000	
	Dissolved Oxygen - mg/l					
0	14.6	13.8	13.0	12.1	11.3	0.017
1	14.2	13.4	12.6	11.8	11.0	0.016
2	13.8	13.1	12.3	11.5	10.8	0.015
3	13.5	12.7	12.0	11.2	10.5	0.015
4	13.1	12.4	11.7	11.0	10.3	0.014
5	12.8	12.1	11.4	10.7	10.0	0.014
6	12.5	11.8	11.1	10.5	9.8	0.014
7	12.2	11.5	10.9	10.2	9.6	0.013
8	11.9	11.2	10.6	10.0	9.4	0.013
9	11.6	11.0	10.4	9.8	9.2	0.012
10	11.3	10.7	10.1	9.6	9.0	0.012
11	11.1	10.5	9.9	9.4	8.8	0.011
12	10.8	10.3	9.7	9.2	8.6	0.011
13	10.6	10.1	9.5	9.0	8.5	0.011
14	10.4	9.9	9.3	8.8	8.3	0.010
15	10.2	9.7	9.1	8.6	8.1	0.010
16	10.0	9.5	9.0	8.5	8.0	0.010
17	9.7	9.3	8.8	8.3	7.8	0.010
18	9.5	9.1	8.6	8.2	7.7	0.009
19	9.4	8.9	8.5	8.0	7.6	0.009
20	9.2	8.7	8.3	7.9	7.4	0.009
21	9.0	8.6	8.1	7.7	7.3	0.009
22	8.8	8.4	8.0	7.6	7.1	0.008
23	8.7	8.3	7.9	7.4	7.0	0.008
24	8.5	8.1	7.7	7.3	6.9	0.008
25	8.4	8.0	7.6	7.2	6.7	0.008
26	8.2	7.8	7.4	7.0	6.6	0.008
27	8.1	7.7	7.3	6.9	6.5	0.008
28	7.9	7.5	7.1	6.8	6.4	0.008
29	7.8	7.4	7.0	6.6	6.3	0.008
30	7.6	7.3	6.9	6.5	6.1	0.008
31	7.5					
32	7.4					
33	7.3					
34	7.2					
35	7.1					

4.3.6 DO-BOD Interactions

The most widely used dissolved oxygen predictive equation is the Streeter-Phelps relationship which predicts the dissolved oxygen concentration downstream from a point source of BOD being discharged into the river. Assuming a constant river cross-sectional area, the dissolved oxygen deficit (C_s-C) can be expressed as:

$$D = D_o \exp \left[\frac{-k_a x}{U}\right] + \frac{L_o k_L}{k_a - k_L} \left[\exp\left(\frac{-k_L x}{U}\right) - \exp\left(\frac{-k_a x}{U}\right)\right] \qquad \text{(IV-18)}$$

where

k_a = reaeration coefficient, 1/day

D_o = initial deficit (at x = 0), mg/l

D = deficit at x, mg/l

L_o = initial BOD (at x = 0), mg/l

k_L = BOD decay coefficient, 1/day

L_o and D_o are found by proportioning BOD and DO deficit concentrations just upstream of the waste discharge with the influx from the discharge itself. As presented earlier in the BOD section, L_o is given by:

$$L_o = \frac{W + L_u Q_u}{Q_w + Q_u} \qquad \text{(IV-19)}$$

where

W = discharge rate of BOD, lb/day

L_u = concentration of BOD in the river upstream of the waste discharge, mg/l

Q_u = river flow rate upstream of discharge, cfs

Q_w = flow rate of waste discharge, cfs

$Q_w + Q_u$ = flow rate of river in the reach under consideration, cfs

W in Equation IV-19 should be expressed in terms of ultimate BOD, and not 5-day BOD for which the loading is often given.

The initial deficit is found from:

$$D_o = C_s - \frac{C_w Q_w + C_u Q_u}{Q_w + Q_u} = \frac{D_w Q_w + D_u Q_u}{Q_w + Q_u} \qquad \text{(IV-20)}$$

where

C_w = concentration of dissolved oxygen in the waste, mg/l

C_u = concentration of dissolved oxygen upstream of the waste discharge, mg/l

D_w = dissolved oxygen deficit in waste, mg/l

D_u = dissolved oxygen deficit upstream, mg/l

In cases where information is lacking, D_o can normally be assumed to be in the range 1-2 mg/l.

If NBOD is to be considered as well as CBOD, Equation IV-18 can be modified as follows:

$$D = D_o \exp\left[\frac{-k_a x}{U}\right] + \frac{L_o k_L}{k_a - k_L}\left[\exp\left(\frac{-k_L x}{U}\right) - \exp\left(\frac{-k_a x}{U}\right)\right]$$

$$+ \frac{N_o k_N}{k_a - k_N}\left[\exp\left(\frac{-k_N x}{U}\right) - \exp\left(\frac{-k_a x}{U}\right)\right] \qquad \text{(IV-21)}$$

If the decay coefficient of NBOD is approximately equal to that of CBOD, Equation IV-18 can be utilized instead of the more complicated Equation IV-21. In this case, L_o in Equation IV-18 is replaced by the sum of L_o and N_o.

4.3.7 Dissolved Oxygen Calculations

Calculation of dissolved oxygen in rivers can proceed as shown in Figure IV-8. The planner needs to estimate the waste loading scheme for the prototype, whether it be a 20 year projection or the current condition. Based on the waste load distribution, the river can then be divided into reaches. By repeated use of Equation IV-18, dissolved oxygen calculations can be performed for one reach after another, starting from some upstream point and proceeding downstream. All data and calculations should be succinctly and clearly recorded to minimize errors.

Equation IV-18 can be solved graphically, as well as analytically, as illustrated in Figure IV-9. The abscissa in Figure IV-9 is proportional to the product of the reaeration coefficient (k_a) and the travel time (t) in days. The following example (Nemerow, 1974) should serve to illustrate the use of the nomogram.

EXAMPLE IV-2

Use of Figure IV-9

Assume that the following information has been calculated:

$k_a = 0.12/\text{day}$

$\frac{k_a}{k_L} = 1.0$

$D_o = 0.77 \text{ mg/l}$

$L_o = 12 \text{ mg/l}$

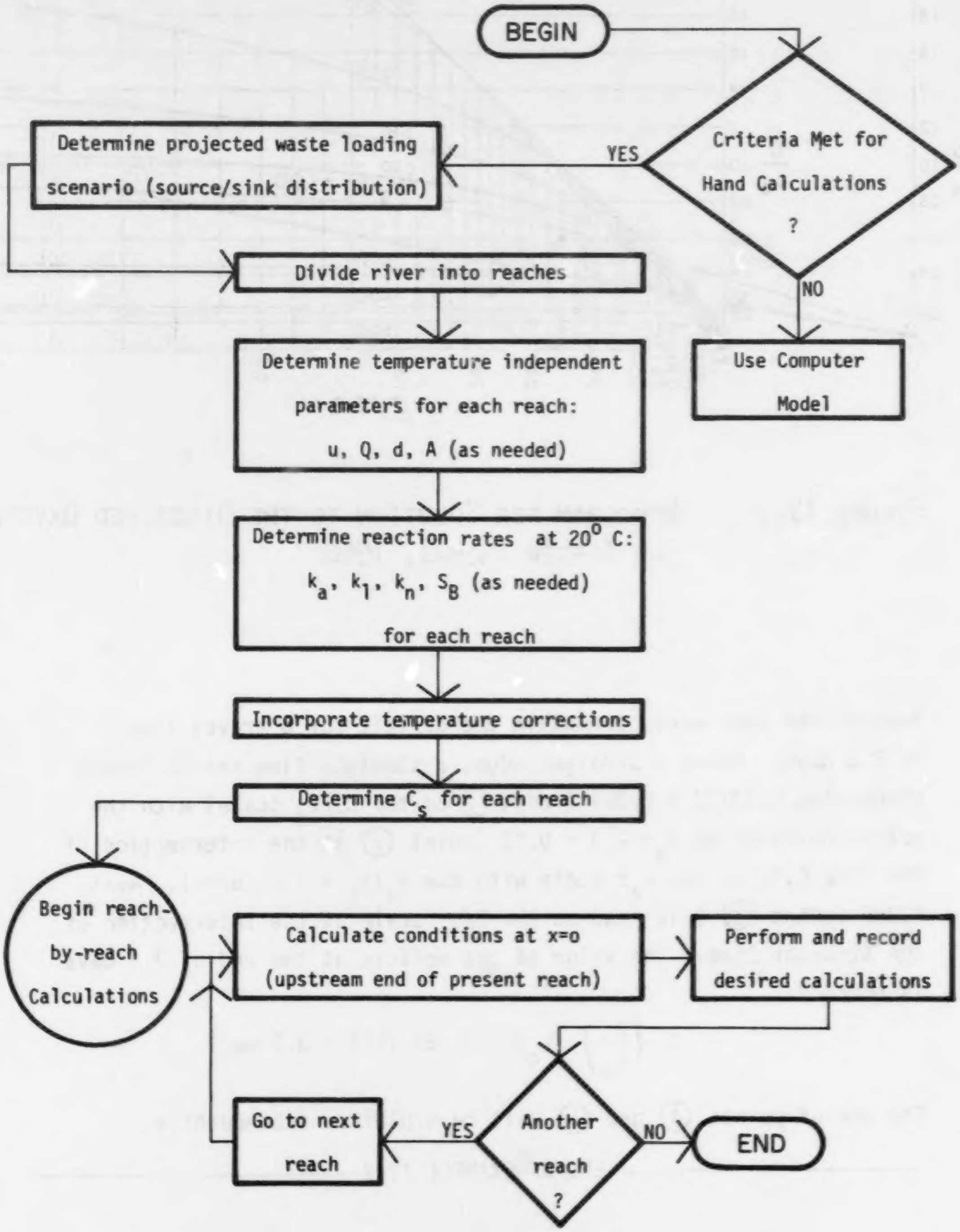

BEGIN
Criteria Met for Hand Calculations ?
YES
NO
Determine projected waste loading scenario (source/sink distribution)
Use Computer Model
Divide river into reaches
Determine temperature independent parameters for each reach: u, Q, d, A (as needed)
Determine reaction rates at 20^{o} C: k_a, k_1, k_n, S_B (as needed) for each reach
Incorporate temperature corrections
Determine C_s for each reach
Begin reach-by-reach Calculations
Calculate conditions at x=o (upstream end of present reach)
Perform and record desired calculations
Another reach ?
YES
NO
Go to next reach
END

FIGURE IV-8 FLOW PROCESS OF SOLUTION TO DISSOLVED OXYGEN PROBLEM IN RIVERS

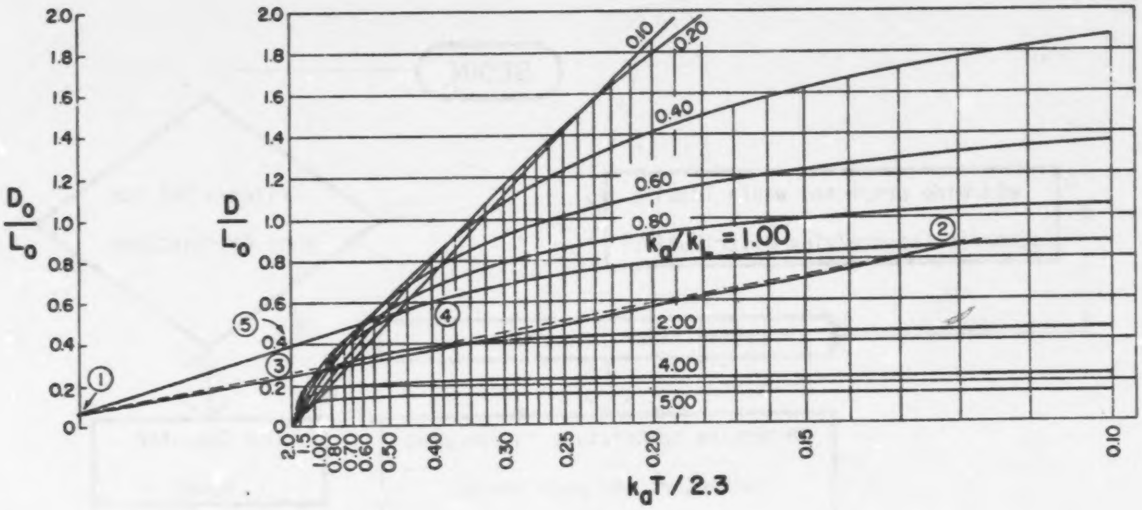


Figure IV-9 Nomogram for Solution to the Dissolved Oxygen Sag (after Thomas, 1948)

Suppose the user wants to obtain the deficit for a travel time of 2.3 days. Using a straight edge, a straight line can be formed connecting 0.77/12 = 0.064 (Point ① on the D_o/L_o scale) with the point representing $k_at/2.3 = 0.12$ (point ② at the intersection of the line 0.12 on the k_at scale with the $k_a/k_L = 1.0$ curve). Next 0.265 (point ③) is read on the D/L_o scale at the intersection of the straight line. The value of the deficit at the end of 2.3 days is then:

$$D = \left(\frac{D}{L_o}\right) (L_o) = 0.265\ (12) = 3.2\ \text{mg/l}$$

The use of points ④ and ⑤ will be explained subsequently.

END OF EXAMPLE IV-2

The dissolved oxygen profile downstream from a waste discharge characteristically has a shape shown in Figure IV-10. If the reach is long enough, the dissolved oxygen will decrease to some minimum value, D_c, at a distance x_c (termed the critical distance). D_c is called the critical deficit. Within any reach there will always be a minimum dissolved oxygen value that occurs, but it may not be the critical deficit, which is defined as the minimum point on a dissolved oxygen sag. The difference between the minimum and critical values should be kept in mind. As one example of the difference between the values, a reach may have a dissolved oxygen profile where concentrations are monotonically decreasing throughout the reach. The minimum DO will then occur at the downstream end of the reach, but this will NOT be the critical DO value, since DO is still decreasing in the downstream direction.

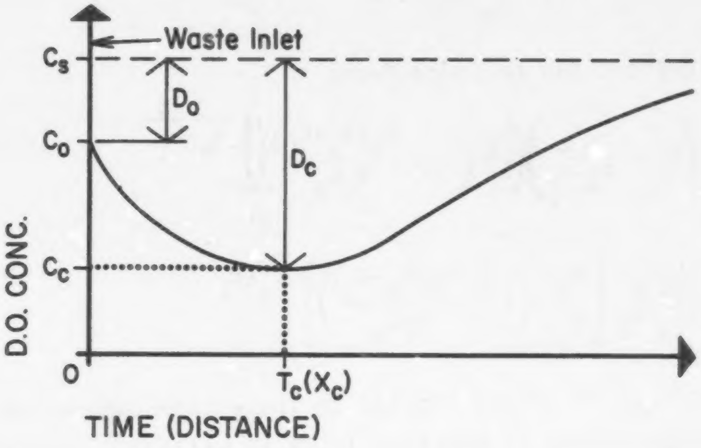


FIGURE IV-10 CHARACTERISTIC DISSOLVED OXYGEN PROFILE DOWNSTREAM FROM A POINT SOURCE OF POLLUTION

The travel time to the critical deficit point is given by:

$$t_c = \frac{1}{k_a - k_L} \ln \left[\frac{k_a}{k_L} \left(1 - \frac{D_o(k_a - k_L)}{k_L L_o} \right) \right] \qquad \text{(IV-22)}$$

The distance downstream can be computed by knowing the travel time and flow velocity:

$$x_c = U \cdot t_c \qquad \text{(IV-23)}$$

The critical deficit can be found from:

$$D_c = \left(D_o - \frac{L_o k_L}{k_a - k_L}\right)\left(\frac{k_a}{k_L}\left(1 - \frac{D_o(k_a - k_L)}{k_L L_o}\right)\right)^{\frac{-k_a}{k_a - k_L}}$$

$$+ \frac{L_o k_L}{k_a - k_L}\left[\frac{k_a}{k_L}\left(1 - \frac{D_o(k_a - k_L)}{k_L L_o}\right)\right]^{\frac{-k_L}{k_a - k_L}} \qquad \text{(IV-24)}$$

Equations IV-22 and IV-24 are limited to those cases where the Streeter-Phelps type relationship of Equation IV-18 is valid. D_c can also be found by solving Equation IV-22 for t_c and then using the nomogram (Figure IV-9) by entering the abscissa at the critical travel time. As a second alternative, use Figure IV-9 exclusively as described in the next example.

EXAMPLE IV-3

Determining Critical Deficit from Figure IV-9

From the previous example (Example IV-2) the dissolved oxygen deficit for a specified travel time was found. Suppose now the critical deficit and the travel time associated with that deficit is desired. Using the information in the previous example ($k_a/k_L = 1$, $D_o/L_o = 0.064$), a straight edge is used to draw a line from point ① to a tangent on the $k_a/k_L = 1$ line.

The point where this line crosses the D/L_o axis (point ⑤) is the maximum deficit that can occur. From the nomogram,

$$\frac{D_c}{L_o} = 0.4$$

$$D_c = (0.4)\ (12) = 4.8 \text{ mg/l}$$

On the k_a/k_L line, point ④ indicates that

$$k_a t/2.3 = 0.4$$

$$t_c = \frac{(0.4)(2.3)}{.12} = 7.7 \text{ days}$$

END OF EXAMPLE IV-3

Solutions to both Equations IV-24 and IV-22 for a broad range of values are presented in Tables IV-10 and IV-11, respectively. Observe that there exist practical limitations to the solutions of both equations, governed by the conditions that the solutions be both positive and real. If in solving Equation IV-22 t_c is negative, the minimum dissolved oxygen concentration actually occurs at $x = 0$, and DO conditions will improve proceeding downstream.

Tables IV-10 and IV-11 are particularly useful for computing the waste assimilative capacity of a river under a given set of conditions. Waste assimilative capacity (WAC), as defined here, is the amount of BOD that can be discharged into a river without causing the minimum dissolved oxygen to go below a desired value. The process of computing this is explained in Example IV-4. In constructing Tables IV-10 and IV-11 more detail was incorporated for D_o/L_o between 0.0 and 0.5. This is because most practical problems fall into this range.
It should be noted that the methods for DO computation first presented differ mainly in degree of resolution and convenience. Selection by the user should be made accordingly.

TABLE IV-10

D_c/L_o VALUES VERSUS D_o/L_o AND k_a/k_L

D_o/L_o	k_a/k_L 0.1	0.3	0.5	0.7	0.9	1.1	1.3	1.5	1.7	1.9	2.1	2.3	2.5	2.7	2.9	3.1	3.3	3.5	3.7	3.9	4.1	4.3	4.5	4.7	4.9
0.00	.77	.60	.50	.44	.39	.35	.32	.30	.28	.26	.24	.23	.22	.21	.20	.19	.18	.17	.17	.16	.15	.15	.14	.14	.14
0.02	.79	.61	.51	.44	.40	.36	.33	.30	.28	.26	.25	.23	.22	.21	.20	.19	.18	.18	.17	.16	.16	.15	.15	.14	.14
0.04	.81	.62	.52	.45	.40	.36	.33	.31	.29	.27	.25	.24	.23	.22	.21	.20	.19	1.8	.17	.17	.16	.16	.15	.15	.14
0.06	.82	.63	.53	.46	.41	.37	.34	.31	.29	.27	.26	.24	.23	.22	.21	.20	.19	.18	.18	.17	.17	.16	.15	.15	.15
0.08	.84	.65	.54	.47	.42	.38	.35	.32	.30	.28	.26	.25	.24	.23	.21	.21	.20	.19	.18	.18	.17	.16	.16	.15	.15
0.10	.85	.66	.55	.48	.43	.39	.36	.33	.31	.29	.27	.26	.24	.23	.22	.21	.20	.19	.19	.18	.17	.17	.16	.16	.15
0.12	.87	.67	.56	.49	.44	.40	.36	.34	.31	.29	.28	.26	.25	.24	.23	.22	.21	.20	.19	.19	.18	.17	.17	.16	.16
0.14	.88	.68	.57	.50	.45	.40	.37	.34	.32	.30	.28	.27	.25	.24	.23	.22	.21	.21	.20	.19	.19	.18	.18	.17	.17
0.16	.90	.69	.58	.51	.45	.41	.38	.35	.33	.31	.29	.27	.26	.25	.24	.23	.22	.21	.21	.20	.19	.19	.18	.18	.17
0.18	.91	.71	.59	.52	.46	.42	.39	.36	.33	.31	.30	.28	.27	.26	.25	.24	.23	.22	.21	.21	.20	.20	.19	.19	.19
0.20	.93	.72	.60	.53	.47	.43	.39	.37	.34	.32	.30	.29	.28	.26	.25	.24	.24	.23	.22	.22	.21	.21	.20	.20	.20
0.22	.95	.73	.62	.54	.48	.44	.40	.37	.35	.33	.31	.30	.28	.27	.26	.25	.25	.24	.23	.23	.22	.22			
0.24	.96	.75	.63	.55	.49	.45	.41	.38	.36	.34	.32	.31	.29	.28	.27	.26	.26	.25	.25	.24	.24				
0.26	.98	.76	.64	.56	.50	.46	.42	.39	.37	.35	.33	.31	.30	.29	.28	.27	.27	.26	.26						
0.28	.99	.77	.65	.57	.51	.47	.43	.40	.38	.36	.34	.32	.31	.30	.29	.29	.28	.28							
0.30	1.01	.78	.66	.58	.52	.48	.44	.41	.39	.37	.35	.34	.32	.31	.31	.30	.30								
0.32	1.03	.80	.67	.59	.53	.49	.45	.42	.40	.38	.36	.35	.34	.33	.32	.32									
0.34	1.04	.81	.68	.60	.54	.50	.46	.43	.41	.39	.37	.36	.35	.34	.34										
0.36	1.06	.82	.70	.61	.55	.51	.47	.44	.42	.40	.38	.37	.36	.36											
0.38	1.07	.84	.71	.62	.56	.52	.48	.45	.43	.41	.40	.39	.38												
0.40	1.09	.85	.72	.63	.57	.53	.49	.46	.44	.42	.41	.40													
0.42	1.11	.86	.73	.65	.58	.54	.50	.47	.45	.44	.43	.42													
0.44	1.12	.88	.74	.66	.60	.55	.51	.49	.47	.45	.44														
0.46	1.14	.89	.76	.67	.61	.56	.53	.50	.48	.47	.46														
0.48	1.15	.90	.77	.68	.62	.57	.54	.51	.49	.48															
0.50	1.17	.92	.78	.69	.63	.59	.55	.53	.51	.50															
0.6	1.25	.99	.84	.76	.69	.65	.62	.60																	
0.7	1.33	1.06	.91	.82	.76	.72	.70																		
0.8	1.41	1.13	.98	.89	.84	.81																			
0.9	1.50	1.20	1.05	.97	.92	.90																			
1.0	1.58	1.27	1.12	1.04	1.00																				
1.1	1.66	1.35	1.20	1.13	1.10																				
1.2	1.75	1.43	1.28	1.21																					
1.3	1.83	1.50	1.36	1.30																					
1.4	1.92	1.58	1.44	1.40																					
1.5	2.00	1.66	1.53																						
1.6	2.09	1.75	1.62																						
1.7	2.17	1.83	1.71																						
1.8	2.26	1.91	1.80																						
1.9	2.34	2.00	1.90																						
2.0	2.43	2.08																							
2.1	2.52	2.17																							
2.2	2.60	2.26																							

D_o/L_o	k_a/k_L 5.1	5.3	5.5	5.7	5.9	6.1	6.3	6.5	6.7	6.9	7.1	7.3	7.5	7.7	7.9	8.1	8.3	8.5	8.7	8.9	9.1	9.3	9.5	9.7	9.9
0.00	.13	.13	.12	.12	.12	.11	.11	.11	.11	.10	.10	.10	.10	.10	.09	.09	.09	.09	.09	.09	.08	.08	.08	.08	.08
0.02	.13	.13	.13	.12	.12	.12	.11	.11	.11	.11	.10	.10	.10	.10	.10	.09	.09	.09	.09	.09	.09	.08	.08	.06	.08
0.04	.14	.13	.13	.13	.12	.12	.12	.11	.11	.11	.11	.10	.10	.10	.10	.10	.09	.09	.09	.09	.09	.09	.08	.08	.08
0.06	.14	.14	.13	.13	.13	.12	.12	.12	.12	.11	.11	.11	.11	.10	.10	.10	.10	.10	.09	.09	.09	.09	.09	.09	.09
0.08	.15	.14	.14	.13	.13	.13	.12	.12	.12	.12	.11	.11	.11	.11	.11	.10	.10	.10	.10	.10	.10	.09	.09	.09	.09
0.10	.15	.15	.14	.14	.14	.13	.13	.13	.12	.12	.12	.12	.11	.11	.11	.11	.11	.11	.11	.10	.10	.10	.10	.10	.10
0.12	.16	.15	.15	.14	.14	.14	.14	.13	.13	.13	.13	.12	.12	.12	.12	.12	.12								
0.14	.16	.16	.16	.15	.15	.15	.14	.14	.14	.14	.14														
0.16	.17	.17	.17	.16	.16	.16																			
0.18	.18	.18	.18																						

TABLE IV-11

$k_a t$ VERSUS D_o/L_o AND k_a/k_L

	k_a/k_L																								
D_o/L_o	0.1	0.3	0.5	0.7	0.9	1.1	1.3	1.5	1.7	1.9	2.1	2.3	2.5	2.7	2.9	3.1	3.3	3.5	3.7	3.9	4.1	4.3	4.5	4.7	4.9
0.00	.26	.52	.69	.83	.95	1.05	1.14	1.22	1.29	1.36	1.42	1.47	1.53	1.58	1.63	1.67	1.71	1.75	1.79	1.83	1.87	1.90	1.93	1.97	2.00
0.02	.25	.51	.68	.82	.93	1.03	1.11	1.19	1.25	1.32	1.37	1.43	1.48	1.52	1.57	1.61	1.65	1.68	1.72	1.75	1.78	1.81	1.84	1.87	1.89
0.04	.25	.50	.67	.80	.91	1.00	1.08	1.16	1.22	1.28	1.33	1.38	1.42	1.47	1.50	1.54	1.57	1.61	1.64	1.66	1.69	1.72	1.74	1.76	1.78
0.06	.25	.50	.66	.79	.89	.98	1.06	1.13	1.18	1.24	1.29	1.33	1.37	1.41	1.44	1.47	1.50	1.53	1.55	1.57	1.59	1.61	1.63	1.65	1.66
0.08	.25	.49	.65	.78	.88	.96	1.03	1.09	1.15	1.20	1.24	1.28	1.31	1.35	1.37	1.40	1.42	1.44	1.46	1.48	1.49	1.50	1.51	1.52	1.53
0.10	.25	.49	.64	.76	.86	.94	1.00	1.06	1.11	1.16	1.19	1.23	1.26	1.28	1.30	1.32	1.34	1.35	1.36	1.37	1.38	1.38	1.38	1.38	1.38
0.12	.24	.48	.63	.75	.84	.92	.98	1.03	1.08	1.11	1.15	1.17	1.20	1.22	1.23	1.24	1.25	1.25	1.26	1.26	1.25	1.24	1.23	1.22	1.20
0.14	.24	.48	.63	.74	.82	.89	.95	1.00	1.04	1.07	1.10	1.12	1.13	1.15	1.15	1.16	1.16	1.15	1.14	1.13	1.11	1.09	1.07	1.04	1.00
0.16	.24	.47	.62	.72	.81	.87	.92	.97	1.00	1.03	1.05	1.06	1.07	1.07	1.07	1.07	1.05	1.04	1.02	.99	.96	.92	.88	.83	.77
0.18	.24	.47	.61	.71	.79	.85	.90	.93	.96	.98	1.00	1.00	1.00	1.00	.99	.97	.95	.92	.88	.84	.79	.73	.66	.57	.48
0.20	.24	.46	.60	.70	.77	.83	.87	.90	.92	.94	.94	.94	.93	.92	.90	.87	.83	.78	.73	.66	.59	.49	.39	.25	.09
0.22	.24	.45	.59	.68	.75	.80	.84	.87	.88	.89	.89	.88	.86	.83	.80	.76	.70	.64	.56	.46	.35	.21	.04		
0.24	.23	.45	.58	.67	.73	.78	.81	.83	.84	.84	.83	.81	.78	.74	.70	.64	.56	.47	.36	.23	.06				
0.26	.23	.44	.57	.66	.72	.76	.79	.80	.80	.79	.77	.74	.70	.65	.59	.50	.41	.28	.13						
0.28	.23	.44	.56	.64	.70	.74	.76	.76	.76	.74	.71	.67	.62	.55	.47	.36	.23	.07							
0.30	.23	.43	.55	.63	.68	.71	.73	.73	.72	.69	.65	.60	.53	.44	.34	.20	.03								
0.32	.23	.43	.54	.62	.66	.69	.70	.69	.67	.64	.59	.52	.44	.33	.20	.02									
0.34	.23	.42	.54	.61	.65	.67	.67	.66	.63	.58	.52	.44	.34	.21	.04										
0.36	.22	.42	.53	.59	.63	.65	.64	.62	.58	.53	.45	.36	.23	.07											
0.38	.22	.41	.52	.58	.61	.62	.61	.58	.54	.47	.38	.27	.12												
0.40	.22	.41	.51	.57	.60	.60	.58	.55	.49	.41	.31	.18													
0.42	.22	.41	.50	.56	.58	.58	.55	.51	.44	.35	.23	.08													
0.44	.22	.40	.49	.54	.56	.55	.52	.47	.39	.29	.15														
0.46	.22	.40	.49	.53	.54	.53	.49	.43	.34	.23	.07														
0.48	.22	.39	.48	.52	.53	.51	.46	.39	.29	.16															
0.50	.21	.39	.47	.51	.51	.48	.43	.35	.24	.09															
0.6	.21	.37	.43	.45	.42	.37	.28	.15																	
0.7	.20	.35	.39	.39	.34	.25	.12																		
0.8	.20	.33	.36	.33	.26	.13																			
0.9	.19	.31	.32	.27	.17	.01																			
1.0	.18	.29	.29	.22	.09																				
1.1	.18	.27	.25	.17	.01																				
1.2	.17	.25	.22	.11																					
1.3	.17	.24	.19	.06																					
1.4	.17	.22	.16	.01																					
1.5	.16	.21	.13																						
1.6	.16	.19	.11																						
1.7	.15	.18	.08																						
1.8	.15	.17	.05																						
1.9	.15	.15	.03																						
2.0	.14	.14																							
2.1	.14	.13																							
2.2	.13	.12																							

	k_a/k_L																								
D_o/L_o	5.1	5.3	5.5	5.7	5.9	6.1	6.3	6.5	6.7	6.9	7.1	7.3	7.5	7.7	7.9	8.1	8.3	8.5	8.7	8.9	9.1	9.3	9.5	9.7	9.9
0.00	2.03	2.06	2.08	2.11	2.14	2.16	2.19	2.21	2.24	2.26	2.28	2.30	2.32	2.35	2.37	2.39	2.41	2.43	2.44	2.46	2.48	2.50	2.52	2.53	2.55
0.02	1.92	1.94	1.97	1.99	2.01	2.03	2.05	2.07	2.09	2.11	2.13	2.15	2.16	2.18	2.20	2.21	2.23	2.24	2.26	2.27	2.28	2.30	2.31	2.32	2.33
0.04	1.80	1.82	1.84	1.86	1.87	1.89	1.90	1.92	1.93	1.94	1.96	1.97	1.98	1.99	2.00	2.01	2.01	2.02	2.03	2.03	2.04	2.05	2.05	2.06	2.06
0.06	1.68	1.69	1.70	1.71	1.72	1.73	1.73	1.74	1.74	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.74	1.74	1.73	1.73	1.72	1.71	1.70
0.08	1.53	1.54	1.54	1.54	1.54	1.54	1.53	1.53	1.52	1.51	1.50	1.49	1.48	1.46	1.45	1.43	1.41	1.39	1.36	1.34	1.31	1.28	1.24	1.21	1.17
0.10	1.37	1.36	1.35	1.34	1.33	1.31	1.29	1.27	1.24	1.22	1.19	1.15	1.11	1.07	1.03	.97	.92	.85	.78	.70	.62	.51	.40	.26	.09
0.12	1.18	1.16	1.13	1.10	1.07	1.03	.99	.94	.88	.82	.75	.67	.58	.47	.35	.21	.03								
0.14	.97	.92	.87	.81	.74	.67	.58	.48	.36	.21	.04														
0.16	.70	.62	.53	.42	.29	.14																			
0.18	.36	.22	.05																						

4.3.8 General Dissolved Oxygen Deficit Equation

The most general dissolved oxygen mass-balance formulation to be presented is as follows:

$$D = \left(\frac{k_L}{k_a - k_L}\right)\left(L_o - \frac{L_{rd}}{k_L}\right)\left[\exp\left(\frac{-j_L}{A_o}\ f(x)\right) - \exp\left(\frac{-j_a}{A_o}\ f(x)\right)\right]$$

$$+ \left(\frac{k_N}{k_a - k_N}\right)\left(N_o - \frac{N_{rd}}{k_N}\right)\left[\exp\left(\frac{-j_N}{A_o}\ f(x)\right) - \exp\left(\frac{-j_a}{A_o}\ f(x)\right)\right]$$

$$+ \frac{R + S_B + L_{rd} + N_{rd} - P}{k_a}\left[1 - \exp\left(\frac{-j_a}{A_o}\ f(x)\right)\right]$$

$$+ D_o \exp\left(\frac{-j_a}{A_o}\ f(x)\right) \qquad \text{(IV-25)}$$

where

P = oxygen production rate due to photosynthesis, mg/l/day

R = oxygen utilization rate due to respiration, mg/l/day

S_B = benthic demand of oxygen, mg/l/day

The distance function f(x) expresses the cross-sectional area relationship throughout the reach. The area can increase or decrease linearly or remain constant. The general form of the relationship is:

$$f(x) = A_o x + \Delta_A\ x^2/_2\ ,\ \Delta_A = \frac{A_f - A_o}{x_L}$$

where

A_f = area at $x = x_L$

A_o = area at $x = 0$

x_L = length of reach

For a reach of constant cross-sectional area, $\Delta_A = 0$.

In developing Equation IV-25 the following relationship for CBOD was used (as originally presented in the BOD section):

$$L = \left(L_o - \frac{L_{rd}}{k_L}\right) \exp\left(\frac{-j_L}{A_o} f(x)\right) + \frac{L_{rd}}{k_L} \qquad \text{(IV-6)}$$

An analogous expression for NBOD was also used.

In Equation IV-25, the distributed sources and sinks (P, R, S_B, L_{rd}, N_{rd}) are all mass fluxes, and no volumetric flow rate is associated with any of these sources and sinks of dissolved oxygen.

4.3.9 Photosynthesis and Respiration

The difficulty of accurately assessing the impact of photosynthesis and respiration on the dissolved oxygen resources of streams is not readily apparent from the single terms appearing in Equation IV-25. Of concern are both free floating and attached algae, as well as aquatic plants. The extent to which algae impact the dissolved oxygen resources of a river is dependent on many factors, such as turbidity, which can decrease light transmittance through the water column. Additionally, the photosynthetic rate constantly changes in response to variations in sunlight intensity. Photosynthetic rates are not truly constant as shown in Equation IV-25. Hence if algal activity is known to be a significant factor affecting the dissolved oxygen balance, the use of a computer model is recommended in order to accurately assess such influences. For example, in the Truckee River in California and Nevada, the diurnal variation of dissolved oxygen has a range of from 150 percent saturation during the daylight hours to 50 percent saturation at night due to algal photosynthesis and respiration, respectively. At the most, hand calculations can only give estimates of net dissolved oxygen production rates that then can be compared to the other source/sink terms in Equation IV-10. From this comparison the significance of each can be estimated.

Table IV-12 presents some observed values of oxygen production by photosynthesis. As shown, dissolved oxygen production is often given in terms of rate per unit area. To convert to units of concentration per unit time, divide by the average depth:

$$P = \frac{\overline{P}}{H}$$

where

$\overline{P}$ = production rate of dissolved oxygen, $gm/m^2/day$

H = average river depth, meters

P = production rate of dissolved oxygen, mg/l/day

P can now be directly compared to other terms in Equation IV-10.

TABLE IV-12

SOME AVERAGE VALUES OF GROSS PHOTOSYNTHETIC PRODUCTION OF DISSOLVED OXYGEN (AFTER THOMANN, 1972 AND THOMAS AND O'CONNELL, 1966)

Water Type	Aver.Gross Production (grams/m^2-day)	Average Respiration (gm/m^2-day)
Truckee River - Bottom attached algae	9	11.4
Tidal Creek - Diatom Bloom ($62\text{-}109.10^6$ diatoms/l)	6	
Delaware Estuary - summer	3-7	
Duwamish River estuary - Seattle, Washington	0.5-2.0	
Neuse River System - North Carolina	0.3-2.4	
River Ivel	3.2-17.6	6.7-15.4
North Carolina Streams	9.8	21.5
Laboratory Streams	3.4-4.0	2.4-2.9

The assimilation number (not the same as assimilative capacity), is defined as the grams of oxygen produced per hour per gram of chlorophyll a. This number may be useful in estimating oxygen production. Table IV-13 gives observed chlorophyll a concentrations and assimilation numbers for various communities. Multiplying the chlorophyll a concentration by the assimilation number yields the oxygen production rate per unit area. A generally low but acceptable estimate of assimilation number is 1 gm oxygen per hour per gram of chlorophyll a.

TABLE IV-13

CHLOROPHYLL _a_ AND ASSIMILATION NUMBER OF VARIOUS COMMUNITIES (AFTER ODUM, _et al_., 1958)

Plankton Communities	Euphotic Zone* Chlorophyll _a_ gm/m^2	Assimilation Number, gm Oxygen Per Hour Per gm Chlorophyll a
(Not including bottom plants)		
Long Island Sound	0.1-0.6	1-3
Diatom bloom, Moriches Bay one meter	0.20	4.5
Sewage Pond, Kadoka, S.D.	1.5	2
Shallow Aquatic Communities with Bottom Plants		
Rocky Mountain Stream, Utah	0.3-1.6	0.7-2.0
Blue-green algal mat, polluted stream, Mission River, Texas, August, 1957	2.5	-

*From the surface to the depth where light intensity = 1% of surface light.

Values of photosynthetic respiration vary widely, ranging from 0.5 $gm/m^2/day$ to greater than 10 $gm/m^2/day$. One suggested relationship between respiration and chlorophyll _a_ is given as (Thomann, 1972):

$$R(mg/l/day) = 0.024\ (\text{chlorophyll } \underline{a})\ (\mu g/l) \quad \text{(IV-26)}$$

where

$$1\ \mu g/l = 10^{-3}\ mg/l$$

Chlorophyll _a_ concentration is most commonly expressed in terms of μg/l.

4.3.10 Benthic Demand

In addition to oxygen utilization by respiration of attached algae, benthic deposits of organic material and attached bacterial growth can utilize dissolved oxygen. Table IV-14 illustrates some uptake rates. As with photosynthesis, the uptake rates are expressed in gm/m^2/day. To use these values in Equations IV-10 or IV-25, division by stream depth (in meters) is necessary. Temperature effects can be approximated by

$$(S_B)_T = (S_B)_{20} \; 1.065^{T-20} \qquad \text{(IV-27)}$$

TABLE IV-14

AVERAGE VALUES OF OXYGEN UPTAKE RATES OF RIVER BOTTOMS (AFTER THOMANN, 1972)

	Uptake (gms O_2/m^2-day) @ 20^{o}C	
Bottom Type and Location	Range	Approximate Average
Sphaerotilus - (10 gm dry wt/M^2)	-	7
Municipal Sewage Sludge - Outfall Vicinity	2-10.0	4
Municipal Sewage Sludge - "Aged" Downstream of Outfall	1-2	1.5
Cellulosic Fiber Sludge	4-10	7
Estuarine mud	1-2	1.5
Sandy bottom	0.2-1.0	0.5
Mineral soils	0.05-0.1	0.07

The areal extent of significant oxygen demanding benthic materials is often limited to the region just below the outfall vicinity. Although the oxygen demand may be great over a short distance, it may be insignificant over larger distances. The response of rivers to areally limited benthic deposits is generally to move the critical deficit upstream, but not to lower its value significantly.

4.3.11 Simplifying Procedures in Dissolved Oxygen Calculations

The usage of Equation IV-25 may be untenable for several reasons, such as lack of available data, or because of the voluminous calculations required to apply it to a large number of reaches. Numerous suggestions are offered that should lead to a simplification of the analysis of the dissolved oxygen problem.

Since the general scope of this section is to facilitate the determination of existing or potential problem areas, the analysis should proceed from a simple approach to the more complicated. It may be adequate to analyze the dissolved oxygen response to only the most severe loadings, neglecting those of secondary importance. If such an analysis clearly indicates dissolved oxygen problems, then the inclusion of any other pollutant discharges would only reinforce that conclusion. More rigorous procedures (e.g., a computer model) could then be employed to perform a detailed analysis.

Suppose the improvement of river dissolved oxygen over a reach due to decreased loading from a point source is of interest. This situation is common, as in the design of waste loading abatement schemes. Such improvement can be estimated by:

$$\Delta D = \Delta D_o \exp\left[\frac{-k_a x}{U}\right] + \left(\frac{k_L}{k_a - k_L}\right)\left(\Delta L_o\right)\left[\exp\left(\frac{-j_L}{A_o} f(x)\right) - \exp\left(\frac{-j_a}{A_o} f(x)\right)\right] \qquad \text{(IV-28)}$$

where

ΔL_o = the change in the initial BOD, mg/l

ΔD = change in deficit in response to ΔL_o

Equation IV-28 was formulated from Equation IV-25 assuming that L_o and D_o are the only changes of significance.

Many rivers have a large number of point sources. Although this is not necessarily a complicating factor, a detailed analysis might be too time consuming for hand calculations. There are several possible alternatives to deal with this situation in order to reduce the number of reaches to be analyzed. The first, already mentioned, is to consider only the significant pollutant sources. Second, as was illustrated in Example IV-1, a number of uniformly distributed point sources can be considered as a single distributed source. Third, combining several adjacent point sources is also possible, if the length of the reach under consideration is long relative to the distance of separation between the point sources. Analogously, a distributed source can be approximated as a point source, contributing the same waste loading and located at the center of the distributed source.

It may be that the planner wants only to determine the critical dissolved oxygen concentration in each of a series of reaches. In this case no more than two values of dissolved oxygen per reach need be calculated. Figure IV-11 shows the solution process to be followed.

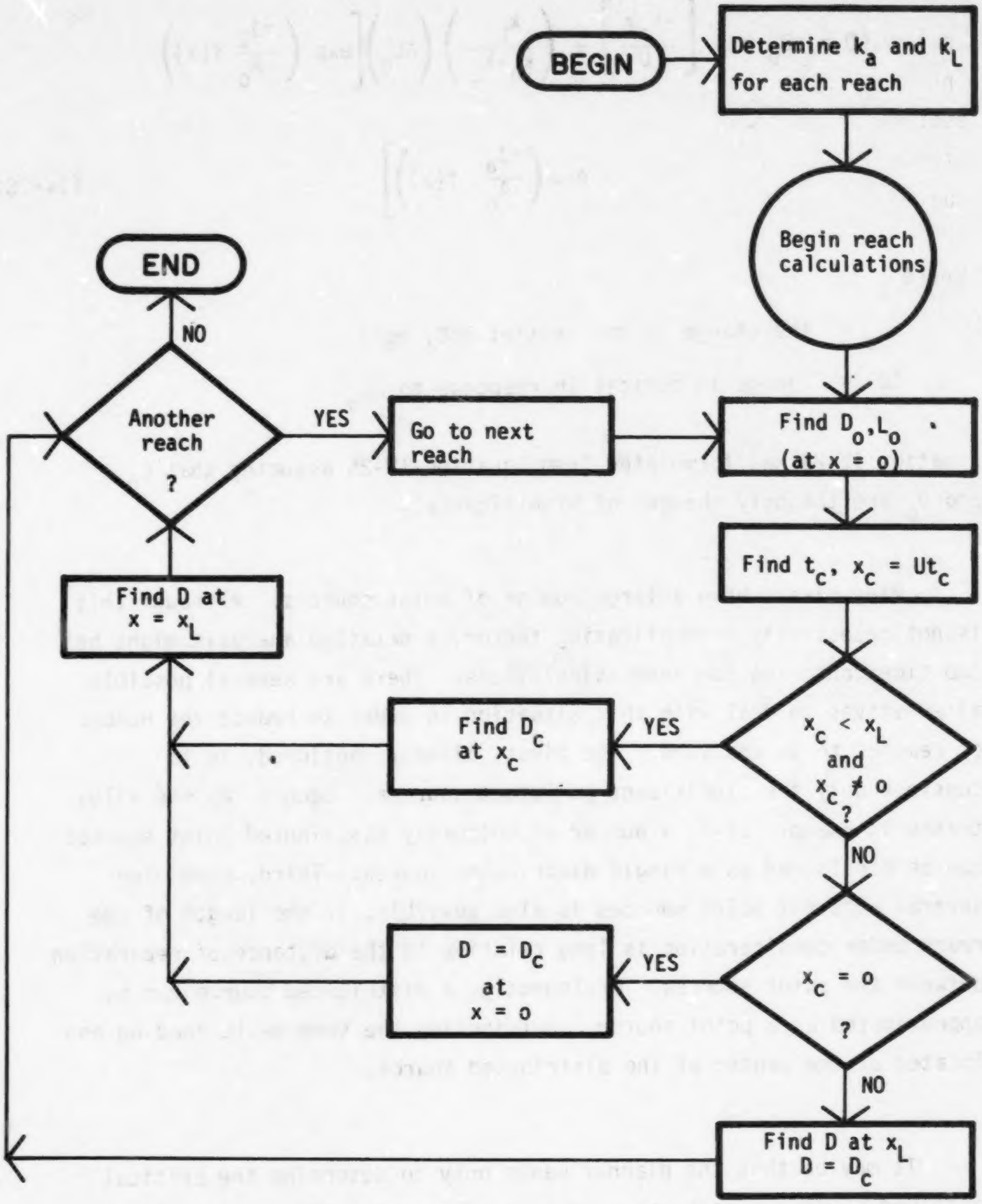

BEGIN
Determine k_a and k_L for each reach
Begin reach calculations
Find D_o, L_o (at x = o)
Find t_c, $x_c = Ut_c$
$x_c < x_L$ and $x_c \neq o$?
YES
Find D_c at x_c
NO
$x_c = o$?
YES
$D = D_c$ at x = o
NO
Find D at x_L $D = D_c$
Find D at $x = x_L$
Another reach ?
YES
Go to next reach
NO
END

FIGURE IV-11 FLOW PROCESS IN REACH BY REACH SOLUTION TO CRITICAL DISSOLVED OXYGEN VALUES

One final note on dissolved oxygen evaluations should be made here. It may be that if the planner is interested only in locating DO problems, he need not perform any computations. This is especially likely where dissolved oxygen data are available at various locations on the river. Plotting dissolved oxygen time trends may reveal when, as well as where, annual dissolved oxygen minima occur.

EXAMPLE IV-4

Determining River Assimilative Capacity from Tables IV-10 and IV-11

Suppose the user wants to determine waste assimilative capacity (WAC) for a river reach that has the following characteristics:

critical dissolved oxygen concentration = 5.0 mg/l (user establishes this)
initial deficit = 1.0 mg/l
average velocity = 0.5 fps
average depth = 4 feet
chloride concentration = 0
temperature range 10^oC to 35^oC

First, k_a and k_L need to be found. From Figure IV-6, k_a (20°) = 0.8/day, and from Figure IV-2, k_L = 0.4/day. At any other temperature then, k_a and k_L can be found from the temperature relationships previously developed:

$$k_a = (k_a)_{20} \quad 1.024^{T-20} \tag{IV-13}$$

$$k_L = (k_L)_{20} \quad 1.047^{T-20} \tag{IV-1}$$

Using Table IV-8 the dissolved oxygen saturation concentration within the temperature range of interest can be found. This information can be then compiled into a table an example of which is shown below (Table IV-15).

TABLE IV-15

COMPILATION OF INFORMATION IN EXAMPLE IV-4

T (^oC)	C_s (mg/l)	C_c (mg/l)	D_c (mg/l)	D_o/D_c	k_a/k_L
10	11.3	5.0	6.3	0.16	2.5
15	10.2	5.0	5.2	0.19	2.2
20	9.2	5.0	4.2	0.24	2.0
25	8.4	5.0	3.4	0.29	1.8
30	7.6	5.0	2.6	0.38	1.6
35	7.1	5.0	2.1	0.48	1.4

Using the values of D_o/D_c and k_a/k_L, L_o can be found, which in this case is the WAC.

Procedure

1. Table IV-15 is entered at the appropriate k_a/k_L column. This is 2.5 at 10°C.

2. Next, the entry within the k_a/k_L column in Table IV-10 is found such that

$$\frac{D_o/L_o}{D_c/L_o} = \frac{D_o}{D_c} = 0.16$$

Since the left-most column of Table IV-10 is D_o/L_o and the entries are D_c/L_o, the ratio of these values is taken until that ratio equals 0.16.

For example, try $D_o/L_o = 0.05$. Then $D_c/L_o = 0.23$ and

$$\frac{0.05}{0.23} = 0.22 > 0.16\text{, too big}$$

try $D_o/L_o = 0.04$. Then $D_c/L_o = 0.23$ and

$$\frac{0.04}{0.23} = .17\text{, close enough}$$

$$\text{then } \frac{D_c}{L_o} = .23\text{, or } L_o = \frac{6.3}{.23} = \underline{27.4}\text{ mg/l}$$

The results are tabulated below for the temperature range 10°C to 35°C.

T(°C)	WAC (mg/l)	D_o/L_o
10	27.4	0.04
15	20.0	0.05
20	15.0	0.07
25	11.3	0.09
30	7.6	0.13
35	5.4	0.19

L_o is directly related to the loading rate of BOD, as expressed earlier in Equation IV-19:

$$\text{WAC} = (L_o)_{\text{critical}} = \frac{L_u Q_u + W_{\text{critical}}}{Q_u + Q_w}$$

From equation IV-19 the critical waste loading W can be found. If desired, this procedure can be repeated for different river flow rates, and WAC and $W_{critical}$ found for the various flows. To do this, different average depths and velocities will be needed. Generally this analysis is most useful carried out under minimum flow conditions, as this is the most critical situation, but higher flows may be of interest to assess the benefits of flow augmentation decisions. Novotny and Krenkel (1975) have used a 20 year, 3-day low flow in analyzing the Holston River in Tennessee. For further discussion of low flow calculations refer to Section 4.4.6.

In interpreting the results of this example the user should be looking more at trends rather than particular results. For example, notice how the WAC decreases with increasing temperature. For every 10° increase the WAC is approximately halved. A similar relationship between WAC and flow rate could also be determined.

Finally, using Table IV-11, the travel time t_c can be determined to the point of critical deficit. The appropriate D_o/L_o and k_a/k_L values are used to find t_c. Table IV-16 illustrates these results.

TABLE IV-16

CRITICAL TRAVEL TIME RESULTS, EXAMPLE IV-4

$T(^oC)$	k_a/k_L	D_o/L_o	$t_c k_a$	k_a	t_c(days)
10	2.5	0.04	1.4	.63	2.2
15	2.2	.05	1.3	.71	1.8
20	2.0	.07	1.2	.8	1.5
25	1.8	.09	1.13	.9	1.2
30	1.6	.13	1.0	1.0	1.0
35	1.4	.19	0.9	1.1	0.8

END OF EXAMPLE IV-4

EXAMPLE IV-5

Critical Deficit Calculations for Multiple Reaches (Illustrating Use of Figure IV-11)

Suppose the critical deficit in each of the three reaches of the river illustrated in Figure IV-12 is to be determined. The conditions upstream of the first discharge are:

$T = 27^{o}C$ depth = 5.0 feet

$Q = 600$ cfs $D_u = 1$ mg/l

$U = 0.4$ fps $L_u = 2$ mg/l

Using these data, along with the solution process outlined in Figure IV-11, the following procedure can be used:

1. Determine k_a, k_L for each reach. For this example it will be assumed that the average depth, velocity, and temperature remain relatively constant over the three reaches, so that k_a and k_L are also the same.

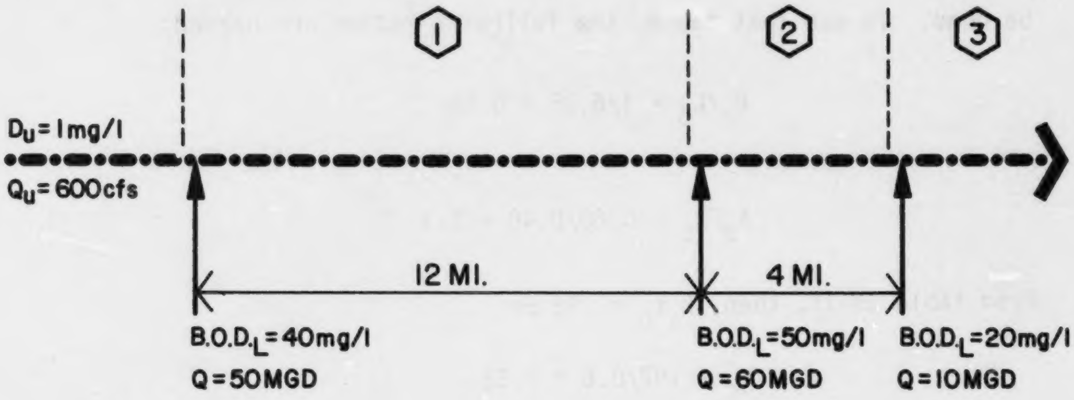

①
②
③
D_U = 1mg/l
Q_U = 600cfs
12 MI.
4 MI.
B.O.D._L = 40mg/l
Q = 50MGD
B.O.D._L = 50mg/l
Q = 60MGD
B.O.D._L = 20mg/l
Q = 10MGD

FIGURE IV-12 HYPOTHETICAL RIVER USED IN EXAMPLE IV-5

$$k_a\ (20) = 0.5, \text{ (from Figure IV-6)}$$

$$k_L\ (20) = 0.35, \text{ (from Figure IV-2)}$$

Using the temperature correction:

$$k_a\ (27) = 0.60, \text{ (from Equation IV-13)}$$

$$k_L\ (27) = 0.48, \text{ (from Equation IV-1)}$$

The saturation dissolved oxygen concentration at 27°C and 0‰ salinity is (from Table IV-9) 8.1 mg/l.

2. For the first reach, calculate L_o and D_o.

$$L_o = \frac{(2)(600)+(40)\ (50)\ (1.55)}{600 + (50)(1.55)} = 6.35 \text{ mg/l}$$

For lack of better information about the dissolved oxygen characteristics of the waste, it can be assumed that $D_o = D_u = 1$ mg/l. The location of where the critical deficit occurs can now be calculated using Table IV-11, or Equation IV-23. In this example Table IV-11 will be used. To use that table, the following ratios are needed:

$$D_o/L_o = 1/6.35 = 0.16$$

and

$$k_a/k_L = 0.60/0.48 = 1.3$$

From Table IV-11, then, $k_a t_c = .92$ or

$$t_c = .92/0.6 = 1.53$$

$$x_c = \frac{(0.4)\ (1.53)\ (3600)\ (24)}{5280} = 10.0 \text{ miles}$$

Since $x_c < 12$, the critical deficit actually exists, and is located 10 miles downstream. From Table IV-10 D_c can be found by entering it with the same ratios used in Table IV-11. The result is:

$$\frac{D_c}{L_o} = .38 \rightarrow D_c = \underline{\underline{2.4}} \text{ mg/l}$$

3. Before the critical conditions in reach 2 can be calculated, the conditions at the upstream end of that reach must be established. The conditions at the downstream end of reach 1 are

$D = 2.3$ mg/l, from Equation IV-18

$L = 2.6$ mg/l from Equation IV-5

The conditions at the upstream end of reach 2 are thus:

$$L_o = \frac{(2.6)\ (677)+(60)\ (1.55)}{677 + 93} = 8.35 \text{ mg/l}$$

$D_o = 2.3$ can be used for lack of better information on the dissolved oxygen concentration in the effluent to reach 2. For use in Table IV-10, it is found that

$$D_o/L_o = .28$$

So

$k_at = .76$

$t_c = .76/0.6 = 1.3$

$x_c = 8.3$ miles

Since reach 2 is only 4.0 miles long, the critical deficit is not reached. Instead the maximum deficit will occur at the downstream end of reach 2, where:

$\underline{D = 3.3\ \text{mg/l}}$, Equation IV-18

$\underline{L = 6.22\ \text{mg/l}}$, Equation IV-5

4. For the beginning of reach 3, L_o and D_o must be found.

$$L_o = \frac{(20)(10)(1.55) + (770.5)\ (6.22)}{770.5 + (10)(1.55)} = 6.5\ \text{mg/l}$$

For D_o, it can be assumed that $C_w = 5.0$ mg/l. From Equation IV-20, then

$$D_o = 8.1 - \frac{(8.1 - 3.3)\ (770.5) + (5.0)(10)\ (1.55)}{770.5 + 15.5} = 3.3\ \text{mg/l}$$

The calculations of critical conditions can now be made for this reach, as done in the previous two.

END OF EXAMPLE IV-5

4.4 TEMPERATURE

4.4.1 Introduction

The biota comprising an established aquatic ecosystem generally respond negatively to significant abnormal temperature fluctuations. Man-induced modifications of rivers and streams can alter the thermal regime, most often by elevating the maximum and mean water temperatures. Repercussions of elevated temperatures are manifested through a shift in the ecological balance and the water quality of rivers. For example, there is a progression in the predominance of algal species from diatoms to green algae to blue-green algae as water temperature increases through a specific range. Thermal discharges can increase the ambient temperature enough to alter the predominant species to the undesirable blue-green algae. Increased metabolic activity of aquatic organisms, such as fish, also accompanies elevated temperature. If the increase is high enough, the results can be lethal. Much data are available today (e.g., Committee on Water Quality Criteria, 1972) delimiting lethal threshold temperatures for aquatic organisms.

Water quality may be adversely affected through decreased solubility of dissolved oxygen and increased biochemical reaction rates. Adequate dissolved oxygen levels, particularly at elevated temperatures, are critical because of the increased metabolic activity. Yet, as previously discussed (see Table IV-9) the saturation concentration of dissolved oxygen diminishes with rising temperature. Worse still, is the concurrent low flow condition which is associated, in many parts of the country, with the warm summer months. For example, in a recent study (EPA, 1974) of 30 river reaches in the U.S., 20 had lower flows in the summer months than in the winter. This situation further reduces assimilative capacity and usually results in the most critical dissolved oxygen levels over the year.

Man can alter the thermal regime of rivers by removing trees (shading), changing the flow regime,and by increasing thermal discharges. Diversions of water from a river can reduce the water depth, and increase the mean and diurnal fluctuation of stream temperature.

In Long Island (Pluhowski, 1968), modification of the natural environment of streams has increased average stream temperatures during the summertime by as much as 9 to 14°F. Concurrent temperature differences of as much as 14 to 18°F between sites on the same stream were observed on days of high solar radiation. A principal factor involved in these occurrences was the removal of vegetation along the banks of the streams, permitting significantly greater penetration of solar radiation. Other contributing factors cited by Pluhowski included increased stormwater runoff, a reduction in the amount of groundwater inflow, and the introduction of ponds and lakes.

4.4.2 Ambient Temperature Increase Due to Thermal Discharge

Figure IV-13 shows an idealization of a run-of-the-river power plant that pumps river water through an intake channel to a condenser, and then discharges the heated effluent directly back into the river. Assuming that the heated effluent has completely mixed with the river water at section A-A, the new ambient river temperature T_f, at that point, can be estimated by:

$$T_f = T_i + \frac{Q_o}{Q_r} \Delta T \qquad \text{(IV-29)}$$

where the terms in Equation IV-29 are defined in Figure IV-13.

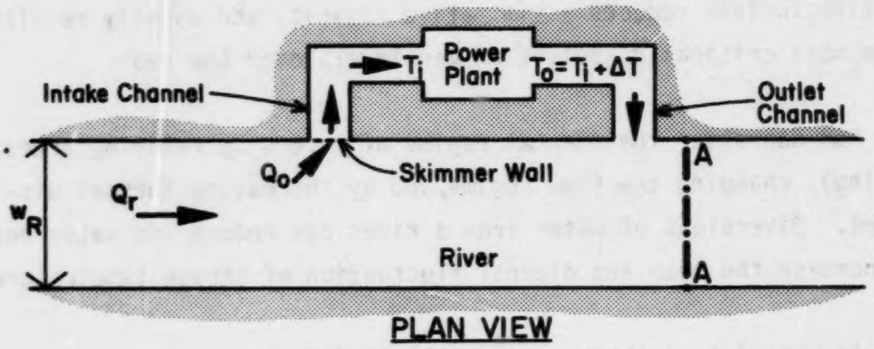


FIGURE IV-13 IDEALIZATION OF A RUN-OF-THE-RIVER POWER PLANT

In many cases the heated effluent may only spread a fraction of the distance across the river. In this case the ambient temperature at A-A is (within that portion of the river affected by the thermal discharge):

$$T_f = \frac{Q_o \left(T_i + \Delta T - \frac{y}{w} T_i\right) + \frac{y}{w} Q_r T_i}{Q_o \left(1 - \frac{y}{w}\right) + \frac{y}{w} Q_r} \qquad \text{(IV-30)}$$

where y/w is the fractional distance across the river over which the thermal plume spreads. Both Equations IV-29 and IV-30 will tend to underestimate the maximum temperature rise because in actuality, the warmer effluent may not mix completely in the vertical direction, but may tend to ride atop the cooler river water.

EXAMPLE IV-6

Estimating ΔT Across a Power Plant Heat Exchange Unit

Suppose the user wants to determine ΔT to use in Equation IV-29 for the Hartford Electric Light Company's South Meadow Steam Electric Power Plant (a fossil fuel plant) located on the Connecticut River. Data available are (Jones, et al., 1975):

capacity . . . . 217 MW
cooling water. . . . . 341 ft^3/sec
waste heat discharged to cooling water . . .422 MW

Since the waste heat being dissipated through the cooling water is known, ΔT can be calculated directly using that value in conjunction with the known Q_o. Assume, however, that the waste heat being discharged is not known. It can be estimated from the plant capacity as follows. First, assume the plant efficiency is 33%. The rate at which fuel is burned when at capacity then, is:

$$\frac{217}{.33} = 658 \text{ MW}$$

If 10% of the total energy is lost up the stacks, then approximately 58% is dissipated through the cooling water, or

$$658\ (.58) = 382 \text{ MW}$$

Compared with the known 422 MW of heat discharged to the cooling water, the above calculation would underestimate ΔT.

ΔT is calculated by

$$\Delta T = \frac{\text{thermal loading rate to cooling water in megawatts}}{\gamma C_p Q_o} \times$$

$$(3.414)\left(10^6 \frac{\text{BTU}}{\text{hr}/_{\text{MW}}}\right)\left(\frac{1 \text{ hr}}{3600 \text{ sec.}}\right)$$

where $\gamma C_p = 62.4$ BTU/ft^3/^{o}F

Q_o = flow rate, ft^3/sec

Substituting the appropriate values into the above equation, it is found that (using the known thermal loading to the cooling water):

$$\Delta T = \frac{(422)\ (3.414)\ (10^6)/3600}{(62.4)\ (341)} = 18.8^o\text{F}$$

END OF EXAMPLE IV-6

4.4.3 Equilibrium Temperature

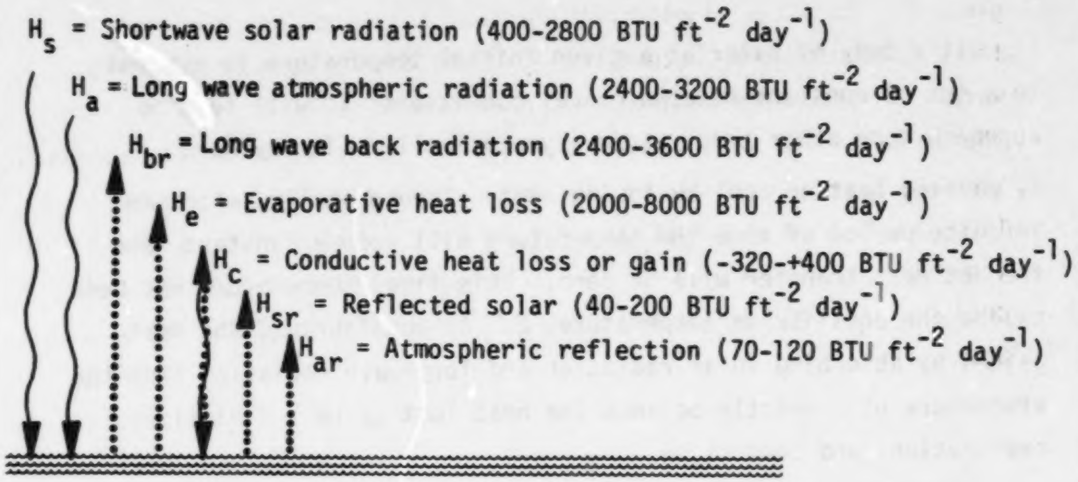
If a body of water at a given initial temperature is exposed to a set of constant meteorological conditions, it will tend to approach some other temperature asymptotically. It may warm by gaining heat or cool by losing heat. Theoretically, after an infinite period of time the temperature will become constant and the net heat transfer will be zero. This final temperature has been called the equilibrium temperature, E. At equilibrium, the heat gained by absorbing solar radiation and long-wave radiation from the atmosphere will exactly balance the heat lost by back radiation, evaporation, and conduction.

These heat fluxes are illustrated in Figure IV-14, which also shows typical ranges for the fluxes. Some of these terms (H_s,H_a,H_{sr}, H_{ar}) are independent of water temperature, while the remainder (H_{br},H_c,H_e) are dependent upon water temperature. At equilibrium then, H_n (net transfer) equals zero, or

$$H_s - H_{sr} + H_a - H_{ar} - H_{br} - H_c - H_e = 0 \qquad \text{(IV-31)}$$

In actuality, the water temperature rarely equals the equilibrium temperature because the equilibrium temperature itself is constantly changing with the local meteorological conditions. The equilibrium temperature will rise during the day when solar radiation is greatest, and fall to a minimum at night when solar radiation is absent.

A daily average equilibrium temperature may be computed using a number of factors including daily average values of radiation, temperature, wind speed, and vapor pressure. The daily average will reach a maximum in midsummer and a minimum in midwinter. Other time periods, such as one month, may also be used for averaging. Since the actual water temperature always tends to approach, but does not reach the equilibrium temperature, it will usually be less than equilibrium


H_s = Shortwave solar radiation (400-2800 BTU ft^{-2} day^{-1})
H_a = Long wave atmospheric radiation (2400-3200 BTU ft^{-2} day^{-1})
H_{br} = Long wave back radiation (2400-3600 BTU ft^{-2} day^{-1})
H_e = Evaporative heat loss (2000-8000 BTU ft^{-2} day^{-1})
H_c = Conductive heat loss or gain (-320-+400 BTU ft^{-2} day^{-1})
H_{sr} = Reflected solar (40-200 BTU ft^{-2} day^{-1})
H_{ar} = Atmospheric reflection (70-120 BTU ft^{-2} day^{-1})

NET RATE AT WHICH HEAT CROSSES WATER SURFACE

$$H_n = \Big[\underbrace{(H_s + H_a - H_{sr} - Har)}_{\substack{\text{Absorbed Radiation } (H_R) \\ \text{Independent of Water Temperature}}} \quad - \quad \underbrace{(H_{br} \pm H_c + H_e)}_{\text{Temperature Dependent Terms}}\Big] \text{BTU ft}^{-2}\text{ day}^{-1}$$

$$H_{br} \sim (T_s + 460)^4$$

$$H_c \sim (T_s - T_a)$$

$$H_e \sim W(e_s - e_a)$$

FIGURE IV-14 MECHANISMS OF HEAT TRANSFER ACROSS A WATER SURFACE (PARKER AND KRENKEL, 1969)

in the spring when temperatures are rising, and greater than equilibrium in the fall when temperatures are dropping. During a one day period, the equilibrium temperature usually varies from above the actual water temperature during the day to below the actual water temperature at night, forcing the water temperature to assume a diurnal cycle.

The amplitude of the actual diurnal water temperature cycle is generally dampened significantly in comparison to the amplitude of the equilibrium temperature cycle due to the large heat capacity of water. A thermal discharge into a water body will usually increase the actual daily amplitude because of the water temperature dependent terms in Equation IV-31, as well as elevate the maximum diurnal temperature. This situation is illustrated in the following example (Edinger, et al., 1968). Figure IV-15 illustrates a flow through cooling pond into which a thermal effluent is discharged (at Station B).

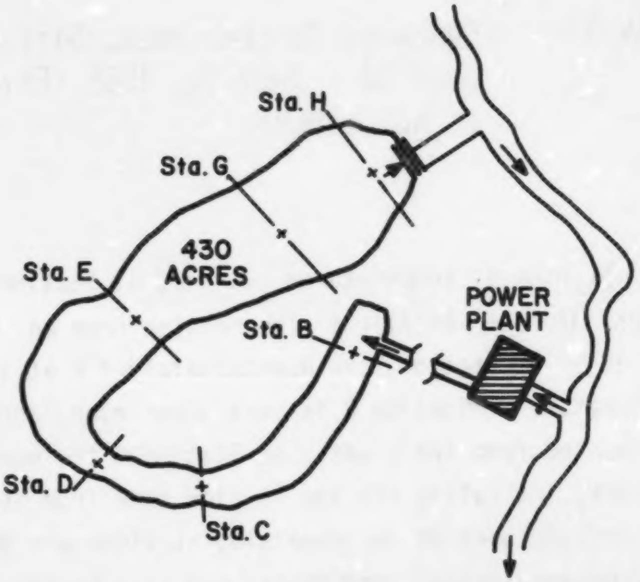


FIGURE IV-15 SCHEMATIC OF SITE NO. 3 COOLING LAKE (FROM EDINGER, ET AL., 1968)

Temperature observations were recorded at Stations B through H at four-hour periods for one week. The findings are depicted in Figure IV-16.

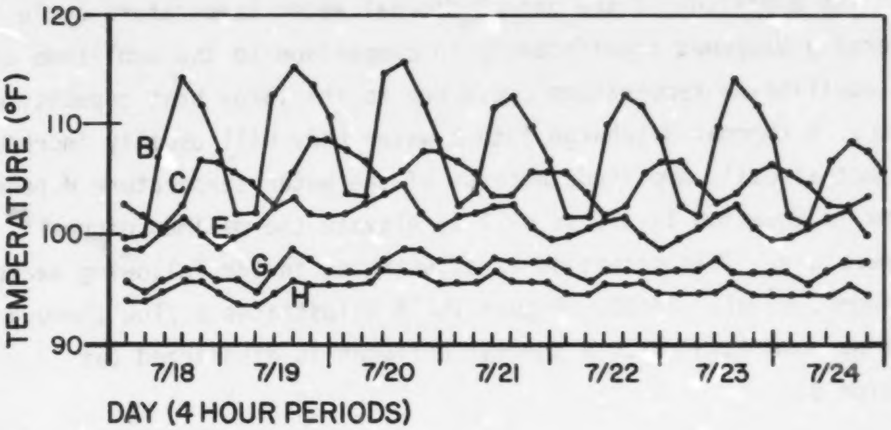


FIGURE IV-16 OBSERVED TEMPERATURES, SITE NO. 3, JULY 18 - JULY 24, 1965 (EDINGER, ET AL., 1968)

Not only are the highest temperatures recorded at Station B, but so are the largest diurnal variations (decreasing from an amplitude of 12.5^{o} at Station B to approximately 2^{o}F at Station H). The peak temperature at Station B is just after noon, corresponding to the peak loading from the plant. At Station C the peak loading is at 1800 hours, indicating the lag in flow time from Station B to C. The peak temperatures at the remaining stations are more influenced by meteorological conditions, and less by the thermal discharge. The relationship of the observed temperatures to the equilibrium temperature over a 24-hour period is shown in Figure IV-17. Note the amplitude of the equilibrium temperature E (33^{o}F amplitude). The average equilibrium temperature, $\overline{E}$, is approximately 91^{o}F. A progression from Station B to Station H indicates that the daily water temperature tends to approach the average equilibrium temperature.

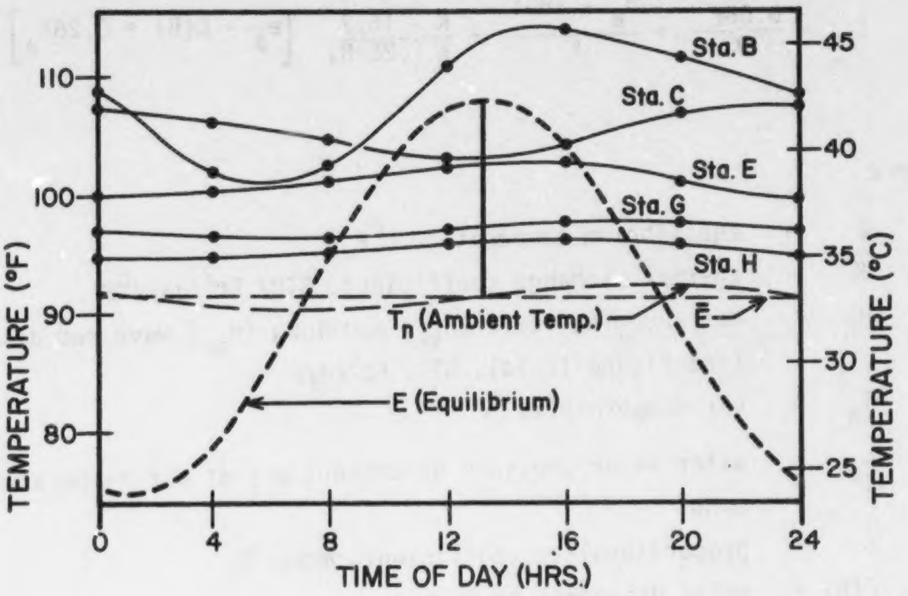


Figure IV-17 Comparison of Computed Equilibrium and Ambient Temperatures with Observed Mean Diurnal Temperature Variations for Site No. 3, July 18-July 24, 1966 (Edinger, et al., 1968)

Stations G and H, and the ambient temperature T_N, all reflect the predominating influence of meteorological conditions. When the water temperature is above the instantaneous equilibrium temperature E, it tends to decrease downward, and when the temperature is below E, it tends to increase. In the early morning and late evening hours, when E is low, the water temperature decreases at these stations. During midday when E is higher, however, the temperatures at these stations increase.

4.4.4 Calculation of Equilibrium Temperature

Studies (Edinger and Geyer, 1965) have shown that the equilibrium temperature of a well mixed body of water can be estimated by:

$$E = -\frac{0.05E^2}{K} + \frac{H_R - 1801}{K} + \frac{K - 15.7}{K\,(.26+B)} \left[e_a - C(B) + 0.26T_a\right] \quad \text{(IV-32)}$$

where

E = equilibrium temperature, °F

K = thermal exchange coefficient, BTU/ ft^2/day/°F

H_R = net incoming short (H_{sn}) and long (H_{an}) wave radiation (see Figure IV-14), BTU/ ft^2/day

T_a = air temperature, °F

e_a = water vapor pressure of ambient air at air temperature, mmHg

B = proportionality coefficient, mmHg/°F

$C(B)$ = value dependent on B, mmHg

The thermal exchange coefficient K is expressed as:

$$K = 15.7 + (0.26+B)\, f(u) \quad \text{(IV-33)}$$

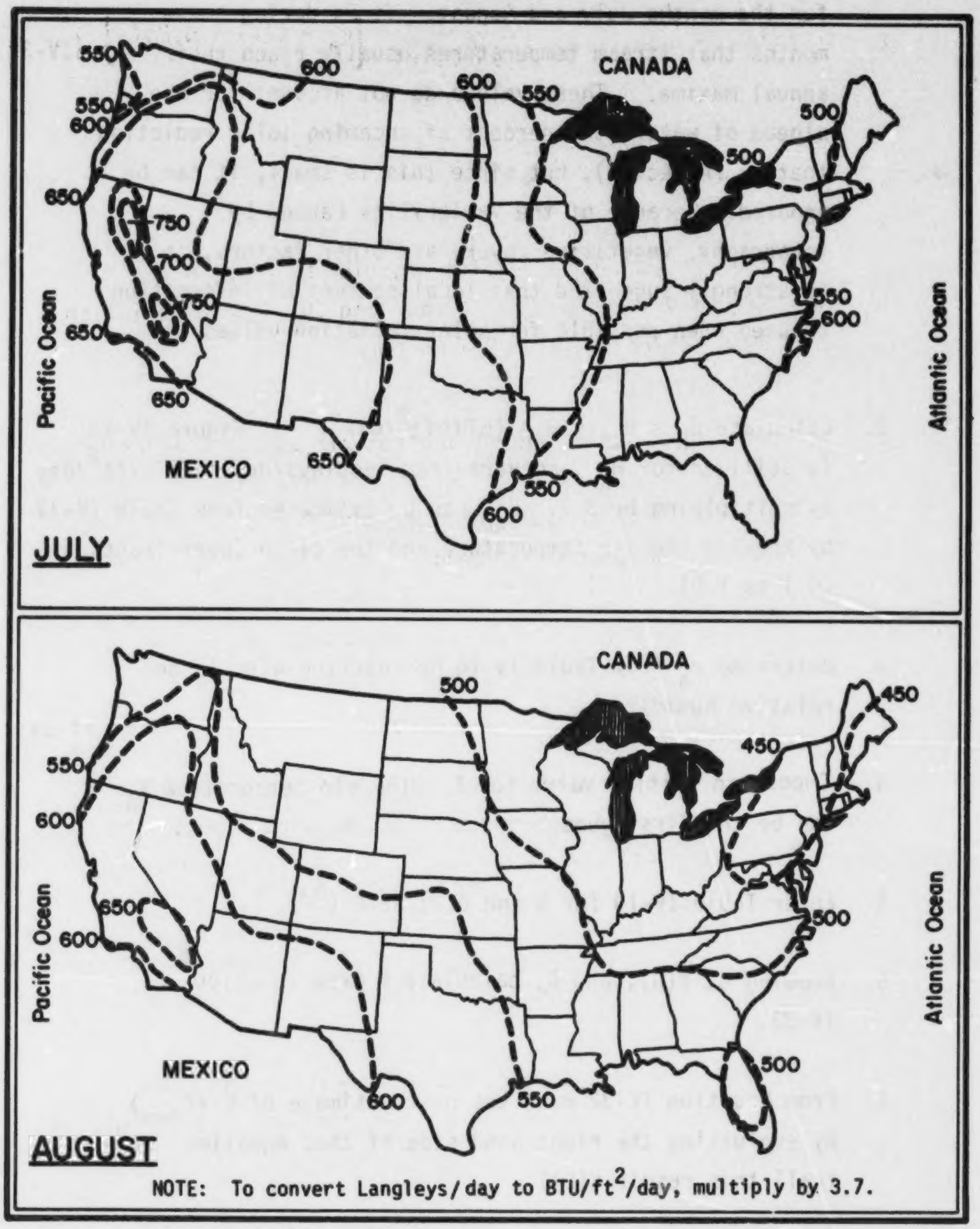
where f(u) is a function of wind speed. Different relationships for f(u) have been developed. For purposes of hand calculations, the following relationship will be used:

$$f(u) = 11.4u$$

where u is the daily average wind speed in mph.

To calculate E using Equation IV-32, an iterative procedure is needed, since K, B, and C(B) depend on E. The following steps outline a solution procedure.

1. Data needed with which to start the procedure include: T_a, relative humidity, wind speed, and net shortwave solar radiation. Figure IV-18 illustrates daily average solar radiation reaching the continental United States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MEXICO
600
550
500
AUGUST

NOTE: To convert Langleys/day to BTU/ft^2/day, multiply by 3.7.

FIGURE IV-18 MEAN DAILY SOLAR RADIATION (LANGLEYS) THROUGHOUT THE U.S. FOR JULY AND AUGUST (U.S. DEPARTMENT OF COMMERCE, 1968)

for the months July and August. It is during these months that stream temperatures usually reach their annual maxima. These values do not account for the albedo of water (the percent of incoming solar radiation that is reflected), but since this is small, it can be ignored. Because of the variability caused by topography, vegetative cover, and other factors, it is strongly suggested that local sources of information be used when possible for solar radiation values.

2. Calculate $H_R = H_{sn} + H_{an}$ (BTU/ ft^2/day). If Figure IV-18 is utilized for H_{sn}, convert from langleys/day to BTU/ft^2/day by multiplying by 3.7. H_{an} can be estimated from Table IV-17 by knowing the air temperature and the cloud cover fraction (0.1 to 1.0).

3. Determine e_a from Table IV-18 by entering with T_a and relative humidity.

4. Choose an initial value for E. The air temperature T_a can be the first guess.

5. Enter Table IV-19 for B and C(B) at E (^oF).

6. Knowing u, f(u), and B, calculate K from Equation IV-33.

7. From Equation IV-32 make the next estimate of E (E_{new}) by evaluating the right hand side of that equation (call this result F(E)).

TABLE IV-17

NET LONG WAVE ATMOSPHERIC RADIATION, H_{an}

Cloud Cover	Temperature (^oF)	H_{an} (BTU/Sq. Ft/Day)	Temperature (^oF)	H_{an} (BTU/Sq. Ft/Day)	Temperature (^oF)	H_{an} (BTU/Sq. Ft/Day)	Temperature (^oF)	H_{an} (BTU/Sq. Ft/Day)	Temperature (^oF)	H_{an} (BTU/Sq. Ft/Day)	Temperature (^oF)	H_{an} (BTU/Sq. Ft/Day)
.1	35	1685	40	1790	45	1900	50	2016	55	2138	60	2266
	65	2400	70	2540	75	2688	80	2842	85	3004	90	3173
.2	35	1694	40	1799	45	1910	50	2026	55	2149	60	2277
	65	2412	70	2553	75	2701	80	2857	85	3019	90	3190
.3	35	1708	40	1814	45	1926	50	2043	55	2167	60	2296
	65	2432	70	2575	75	2724	80	2881	85	3045	90	3216
.4	35	1728	40	1835	45	1949	50	2067	55	2192	60	2323
	65	2461	70	2605	75	2756	80	2914	85	3080	90	3254
.5	35	1754	40	1863	45	1978	50	2098	55	2225	60	2358
	65	2497	70	2644	75	2797	80	2958	85	3126	90	3303
.6	35	1785	40	1896	45	2013	50	2136	55	2265	60	2400
	65	2542	70	2691	75	2847	80	3011	85	3182	90	3362
.7	35	1822	40	1936	45	2055	50	2180	55	2312	60	2450
	65	2595	70	2747	75	2907	80	3074	85	3249	90	3432
.8	35	1865	40	1981	45	2103	50	2232	55	2366	60	2508
	65	2656	70	2812	75	2975	80	3146	85	3325	90	3513
.9	35	1914	40	2033	45	2158	50	2290	55	2428	60	2573
	65	2725	70	2885	75	3053	80	3228	85	3412	90	3604
1.0	35	1968	40	2091	45	2220	50	2355	55	2497	60	2646
	65	2803	70	2967	75	3139	80	3320	85	3509	90	3707

TABLE IV-18

WATER VAPOR PRESSURE (mmHg) VERSUS AIR TEMPERATURE, T_a, AND RELATIVE HUMIDITY

T_a	e_s*	RELATIVE HUMIDITY									
(°F)	(mmHg)	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
35	5.2	0.5	1.0	1.6	2.1	2.6	3.1	3.6	4.2	4.7	5.2
40	6.3	0.6	1.3	1.9	2.5	3.2	3.8	4.4	5.0	5.7	6.3
45	7.6	0.8	1.5	2.3	3.0	3.8	4.6	5.3	6.1	6.8	7.6
50	9.1	0.9	1.8	2.7	3.6	4.6	5.5	6.4	7.3	8.2	9.1
55	11.0	1.1	2.2	3.3	4.4	5.5	6.6	7.7	8.8	9.9	11.0
60	13.1	1.3	2.6	3.9	5.2	6.6	7.9	9.2	10.5	11.8	13.1
65	15.6	1.6	3.1	4.7	6.2	7.8	9.4	10.9	12.5	14.0	15.6
70	18.6	1.9	3.7	5.6	7.4	9.3	11.2	13.0	14.9	16.7	18.6
75	22.0	2.2	4.4	6.6	8.8	11.0	13.2	15.4	17.6	19.8	22.0
80	26.0	2.6	5.2	7.8	10.4	13.0	15.6	18.2	20.8	23.4	26.0
85	30.5	3.1	6.1	9.2	12.2	15.3	18.3	21.4	24.4	27.5	30.5
90	35.8	3.6	7.2	10.7	14.3	17.9	21.5	25.1	28.6	32.2	35.8
95	41.8	4.2	8.4	12.5	16.7	20.9	25.1	29.3	33.4	37.6	41.8
100	48.7	4.9	9.7	14.6	19.5	24.4	29.2	34.1	39.0	43.8	48.7

* e_s = saturated vapor pressure

TABLE IV-19

B AND C(B) AS FUNCTIONS OF TEMPERATURE

Temperature (^{o}F)	B (mmHg/^{o}F)	C(B) (mmHg)	Temperature (^{o}F)	B (mmHg/^{o}F)	C(B) (mmHg)
45	.286	-5.5	70	.660	-22.9
46	.296	-4.5	71	.680	-23.6
47	.306	-4.1	72	.701	-24.4
48	.317	-4.2	73	.722	-25.4
49	.328	-4.6	74	.743	-26.5
50	.340	-5.4	75	.765	-27.8
51	.352	-6.3	76	.787	-29.3
52	.365	-7.5	77	.810	-31.0
53	.378	-8.7	78	.833	-33.0
54	.391	-10.0	79	.857	-35.1
55	.405	-11.2	80	.881	-37.6
56	.419	-12.5	81	.905	-40.3
57	.433	-13.6	82	.930	-43.2
58	.448	-14.7	83	.955	-46.4
59	.464	-15.8	84	.980	-49.7
60	.479	-16.7	85	1.006	-53.3
61	.496	-17.6	86	1.033	-57.1
62	.512	-18.3	87	1.060	-61.0
63	.529	-19.0	88	1.087	-64.9
64	.547	-19.6	89	1.114	-68.9
65	.564	-20.1	90	1.142	-72.9
66	.583	-20.7	91	1.171	-76.7
67	.601	-21.2	92	1.200	-80.4
68	.620	-21.7	93	1.229	-83.8
69	.640	-22.3	94	1.259	-86.8
			95	1.289	-89.3

8. The next estimate of E is:

$$E_{new} = 0.3E + 0.7\ F(E)$$

(Note: this choice of E_{new} brings about a more rapid convergence to the answer than would use of E alone).

9. If $|E_{new} - E| \leq 1^{o}F$, then $E_{actual} = E_{new}$.
If $|E_{new} - E| > 1^{o}F$, return to step 5 with E_{new} and repeat procedure until convergence criterion is met, namely, $E_{actual} = E_{new}$.

Instantaneous, daily average, weekly average, or even longer term average equilibrium temperature, $\overline{E}$, can be calculated by using mean meteorological conditions over the period of interest and following the solution procedure just outlined. The daily equilibrium temperature is a useful concept because it is the temperature about which the instantaneous temperature tends to oscillate. Calculating the daily average $\overline{E}$ under the most crucial annual meteorological conditions (usually occurring in July or August) yields the highest temperature about which that water body tends to naturally oscillate. The repercussions of man's activities in terms of altering $\bar{E}$ can thus be estimated and analyzed for potential impact.

EXAMPLE IV-7

Calculation of Equilibrium Temperature

On Long Island, New York, studies done by Pluhowski (1968) have indicated that shading of streams by a natural vegetative canopy can drastically affect the shortwave solar radiation reaching those streams. The results of some of his findings are presented in Table IV-20. In the summer, when leaves are on the trees, the actual solar radiation reaching the Connetquot River can be as low as 29% of that reaching unobstructed sites at nearby Mineola or Brookhaven.

TABLE IV-20

SUMMARY OF SOLAR-RADIATION DATA
FOR MINEOLA, BROOKHAVEN, AND THE CONNETQUOT RIVER SITES

Solar Site	Dates	Mean-Daily Solar Radiation in Langleys: for the Indicated Periods				Ratio= Connetquot River Observed / Connetquot River Unobstructed
		Mineola	Brookhaven	Connetquot River Estimated	Connetquot River Observed	
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	Jan. 30, 31, 1967	235	244	240	148	0.62
2	Jan. 28, 29, 1967	148	130	137	96	.70
3	Jan. 25, 26, 1967	135	135	135	104	.77
1	Apr. 21-23, 1967	466	464	465	343	.74
	Apr. 16-18, 1968	452	502	502	389	.77
2	Apr. 19, 20, 1967	436	386	429	384	.90
3	Apr. 24-26, 1967	408	411	410	401	.98
1	June 9-11, 1967	600	599	599	254	.42
2	June 7, 8, 1967	664	671	669	531	.79
3	June 12-14, 1967	527	523	525	443	.84
1	Aug. 26-28, 1967	275	260	266	78	.29
2	Aug. 22-24, 1967	277	328	308	162	.53
3	Aug. 29, 30, 1967	504	484	492	338	.69
1	Nov. 28, 29, 1967	204	-	204	86	.42

Note 1 - Radiation data in column 5 are estimated unobstructed horizon values for Connetquot River based on data from Mineola and Brookhaven (cols. 3,4).

Note 2 - Solar site 1 is typically heavily forested, solar site 2 is moderately to heavily forested, and solar site 3 is moderately forested.

Suppose the user is interested in calculating how the removal of the vegetative cover might affect $\bar{E}$. Consider the period 22-24 August, 1967, when the Connetquot River received 162 langleys/day of a possible 308 langleys/day of shortwave solar radiation. Representative meteorological conditions during this time were:

$T_a = 65^oF$
$u = 2$ mph
cloud cover fraction = 0.5
relative humidity = 80%

The steps in solving for $\bar{E}$ are as follows:

1. Data have been gathered, as previously listed.
2. $H_{sn} = 162\ (3.7) = 600$ BTU/ ft^2/day. This value assumes that the vegetation canopy blocks 47% of the solar radiation. From Table IV-17, H_{an} is (.5 cloud cover at 65^oF) 2497 BTU/ ft^2/day. Thus,
$$H_R = 2497 + 600 = 3097 \text{ BTU/ } ft^2\text{/day.}$$
3. At 80% relative humidity and an air temperature of 65^oF, $e_a = 12.5$ mmHg from Table IV-18.
4. As an initial guess of E, assume $E_1 = 65^oF$, the air temperature.
5. From Table IV-19, B = .56, C(B) = -20.1
6. $K = 15.7 + (.26 + .56)\ (11.4)\ (2) = 34.4$

7. $$F(E_1) = \frac{-0.05(65)^2}{34.4} + \frac{3098-1801}{34.4} + \frac{34.4-15.7}{34.4(.26+.56)}$$

$$\times \left[12.5 + 20.1 + .26(65)\right] = -6.1 + 37.7 + 33.0 = 64.6$$

8. $E_2 = .3(65) + .7(64.6) = 64.7$

9. Since $|E_2-E_1|\ 1^{O}F$ $\qquad \underline{\underline{\bar{E} = 64.7^{O}F}}$

Now suppose the user wants to find $\overline{E}$ for no reduction in H_{sn} due to shading. Steps 1 through 9 again are repeated, using H_{sn} = 308(3.7) = 1140 BTU/ft^2/day, with otherwise the same meteorological conditions. Without detailing the calculations here, it is found that $\overline{E}$ = 73.7^{O}, a 9^{O}F increase.

It is evident then that altering the solar radiation penetrating to the stream can significantly change $\overline{E}$. Even more severe cases of repression of shortwave radiation (as noted by the 71% reduction on 26-28 August, 1967, Table IV-20) are possible, exemplifying the large differences which may be observed.

END OF EXAMPLE IV-7

To estimate the effects of shading over the day, the incoming solar radiation should be calculated, first assuming no shading, but otherwise using existing meteorological conditions for the time of the year of interest. The effects of shading should be superimposed upon this result as a percent reduction. The following (Pluhowski, 1968) can serve as guidelines in estimating solar radiation reduction:

- 0-25% reduction: shading generally restricted to early morning and late afternoon.

- 25-50% reduction: some sunshine penetration in morning and evening. Considerable sunshine between 1000 and 1400 hours.

- 50-75% reduction: very little sunshine penetration in morning or late afternoon. Some sunshine between 1000 and 1400 hours.

- Greater than 75% reduction: very little penetration even at noon.

If the diurnal variation of E is desired (such as given in Figure IV-17), it can be estimated by a sinusoidal curve:

$$E = \frac{E_{max} + E_{min}}{2} - \left(\frac{E_{max} - E_{min}}{2} \right) \cos \frac{2\pi T}{24} \qquad \text{(IV-34)}$$

where

E_{max} = maximum daily equilibrium temperature (occurs at noon, solar time)

E_{min} = minimum daily equilibrium temperature (occurs at approximately midnight)

T = time of day, from 0 to 24 hours

Both E_{max} and E_{min} can be found by applying Equation IV-32 for the instantaneous meteorological conditions occurring at noon and midnight, respectively. The maximum instantaneous solar shortwave radiation can be estimated by:

$$(H_{sn})_{max} = \frac{(12)\ (3.14)}{N}\ \bar{H}_{sn} \qquad \text{(IV-35)}$$

where

$\bar{H}_{sn}$ = average daily shortwave solar radiation for the day under consideration

N = number of daylight hours, from sunrise to sunset

The shortwave radiation at midnight is, of course, for all practical purposes zero.

4.4.5 Longitudinal Temperature Variation

If the temperature at a particular location in a river is known, the steady-state temperature distribution downstream from that point can be estimated by:

$$\frac{T - E}{T_m - E} = \exp\left(\frac{-Kx}{\rho C_p U d}\right) \qquad \text{(IV-36)}$$

where

T_m = temperature at x = 0, ^oF

T = stream temperature at a distance x

E = equilibrium temperature, ^oF

K = thermal transfer coefficient, BTU/ft^2/day/^oF

U = stream velocity, ft/sec

d = stream depth, feet

ρ = water density, lb/ft^3

C_p = heat capacity of water, BTU/lb/^oF (ρC_p = 62.4 BTU/ft^3/^oF)

Since the equilibrium temperature E is constantly changing, questions arise as to which equilibrium temperature should be used in Equation IV-36, and what elapsed time is acceptable. In general the time elapsed (i.e., the travel time) should not exceed the time base for which E is calculated. For example, if an average daily E is calculated and used in Equation IV-36, then this equation should be restricted to analyzing cases where the travel time is less than one day. If it is desired to determine the temperature distribution downstream from a thermal discharge in the afternoon hours, Equation IV-34 can be used to calculate an E valid for the afternoon in question.

An important fact is revealed upon inspection of Equation IV-36. Suppose that a thermal discharge heats the ambient water to a temperature T_m, but T_m is less than the instantaneous equilibrium temperature E. In that instance the stream temperature will continue to rise exponentially downstream, approaching E. The rate at which T approaches E is dependent on the thermal transfer coefficient, as well as stream velocity and depth. Equation IV-36 is graphically illustrated in Figure IV-19.

EXAMPLE IV-8

Use of Figure IV-19

Suppose an average daily thermal transfer coefficient, K, of 200 BTU/ft^2/day has been calculated. The river of interest has an initial temperature "excess" (i.e., T_m-E>0). How far downstream will that excess be 50% of the original? Other stream data:

$U = .5$ fps
$d = 4$ feet
$\rho C_p = 62.4$ BTU/ft^3/^oF

From Figure IV-19, r is to be found (from Figure IV-15) such that

$$\frac{T - E}{T_m - E} = .5$$

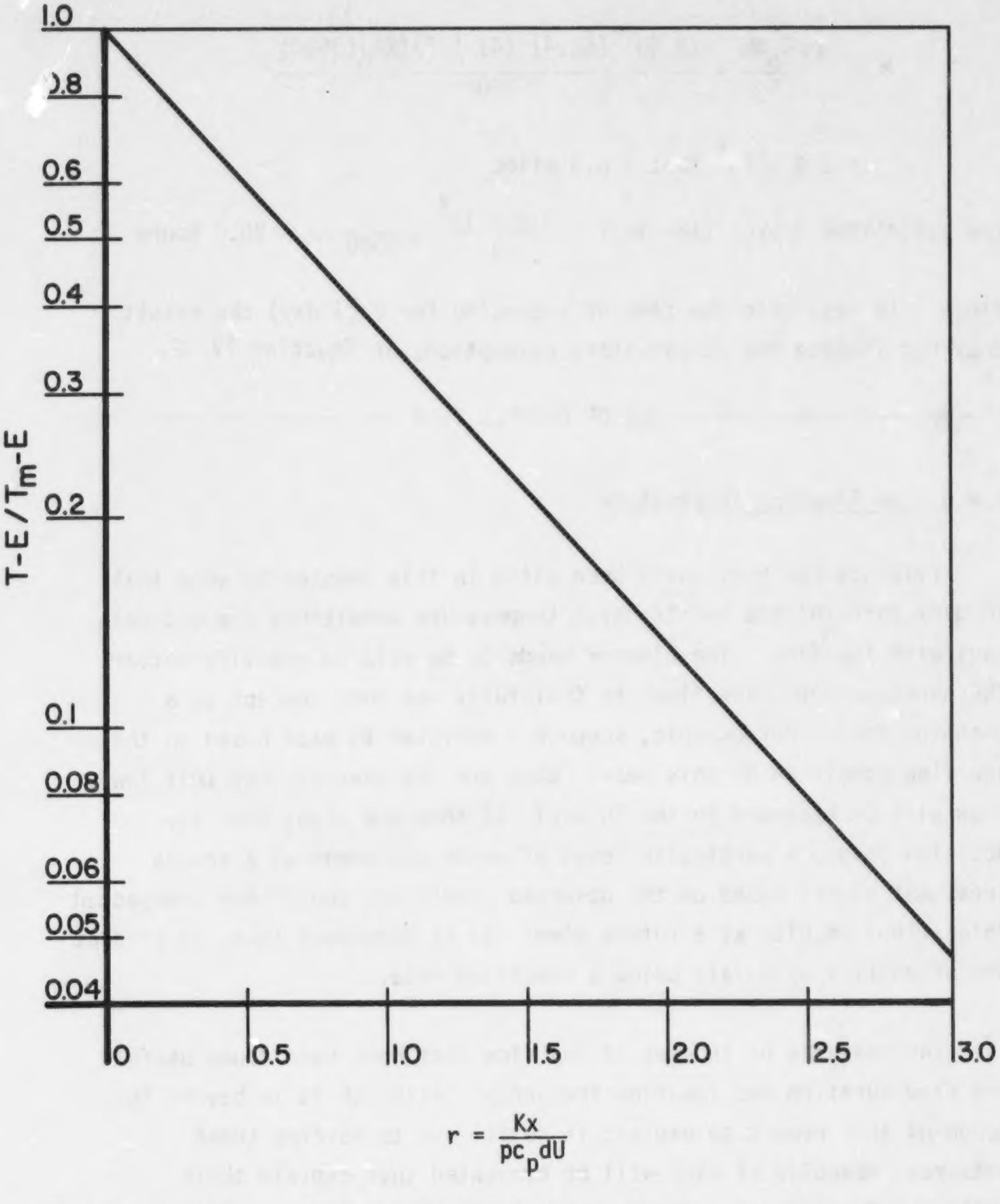

FIGURE IV-19 DOWNSTREAM TEMPERATURE PROFILE FOR COMPLETELY MIXED STREAM, T-E/T$_M$-E VS. r (FROM EDINGER, 1965)

The correct r equals 0.68. Solving for x in terms of r it is found:

$$x = \frac{r \rho C_p dU}{K} = \frac{(0.68)\ (62.4)\ (4)\ (.5)(24)(3600)}{200}$$

$$= 3.6 \times 10^4 \text{ feet} = 6.9 \text{ miles}$$

The associated travel time is $T = \frac{3.6 \times 10^4}{.5} \times \frac{1}{3600}$ hr = 20.4 hours

Since T is less than the time of averaging for E (1 day) the result does not violate the steady-state assumptions of Equation IV-36.

——————— END OF EXAMPLE IV-8 ———————

4.4.6 Low Flow and Temperature

Evidence has previously been cited in this chapter to show that in many parts of the country high temperature conditions are concomitant with low flow. The planner needs to be able to quantify better the nebulous term "low flow" to fruitfully use this concept as a planning tool. For example, suppose a decision is made based on the low flow condition of this year. What are the chances that this low flow will be exceeded in the future? If they are high, then any decision (e.g., a particular level of waste abatement at a sewage treatment plant) based on the observed conditions could have unexpected deleterious results at a future time. It is paramount then, to predict how often flow will fall below a specified rate.

Two measures or indexes of low flow that have been found useful are flow duration and low-flow frequency. Although it is beyond the scope of this report to explain in detail how to develop these measures, examples of each will be presented that explain their utility. The majority of the material in this section is from

Cragwall (1966) who provides a discussion on low flow, and cites additional references. Many texts on engineering hydrology (e.g., Linsley et al., 1958) also discuss low flow.

Figure IV-20 shows a flow duration curve for the Hatchie River at Bolivar, Tennessee. The vertical axis is the daily discharge and the horizontal is the percent of time a flow is equaled or exceeded. For example, 95 percent of the time from 1930-58 the flow exceeded 177 cfs. It can also be assumed that this flow (177 cfs) will probably be exceeded 95% of the time in other years. Thus this concept offers one means by which to quantify "low flow".

A second concept is the low flow frequency curve, illustrated in Figure IV-21. This depicts the relationship between discharge and recurrence interval of different duration flows. For example the 7 day mean flow of 100 cfs can be expected to occur once each 19 years. Stated another way, since probability is the reciprocal of recurrence interval, in any one year there is about a 5 percent probability that a seven day mean flow of less than 100 cfs will occur. A commonly used flow for analyses is the 7 day mean flow at a recurrence interval of 10 years, or $7Q_{10}$.

4.5 NUTRIENTS AND EUTROPHICATION POTENTIAL

4.5.1 Introduction

Within the past few years the elements most often responsible for accelerating eutrophication - nitrogen and phosphorus - have shown generally increasing levels in rivers (EPA, 1974). Median concentrations increased in the period from 1968 to 1972 over the period from 1963 to 1967 in 82 percent of the reaches sampled for total phosphorus, 74 percent for nitrate, and 56 percent for total phosphate.

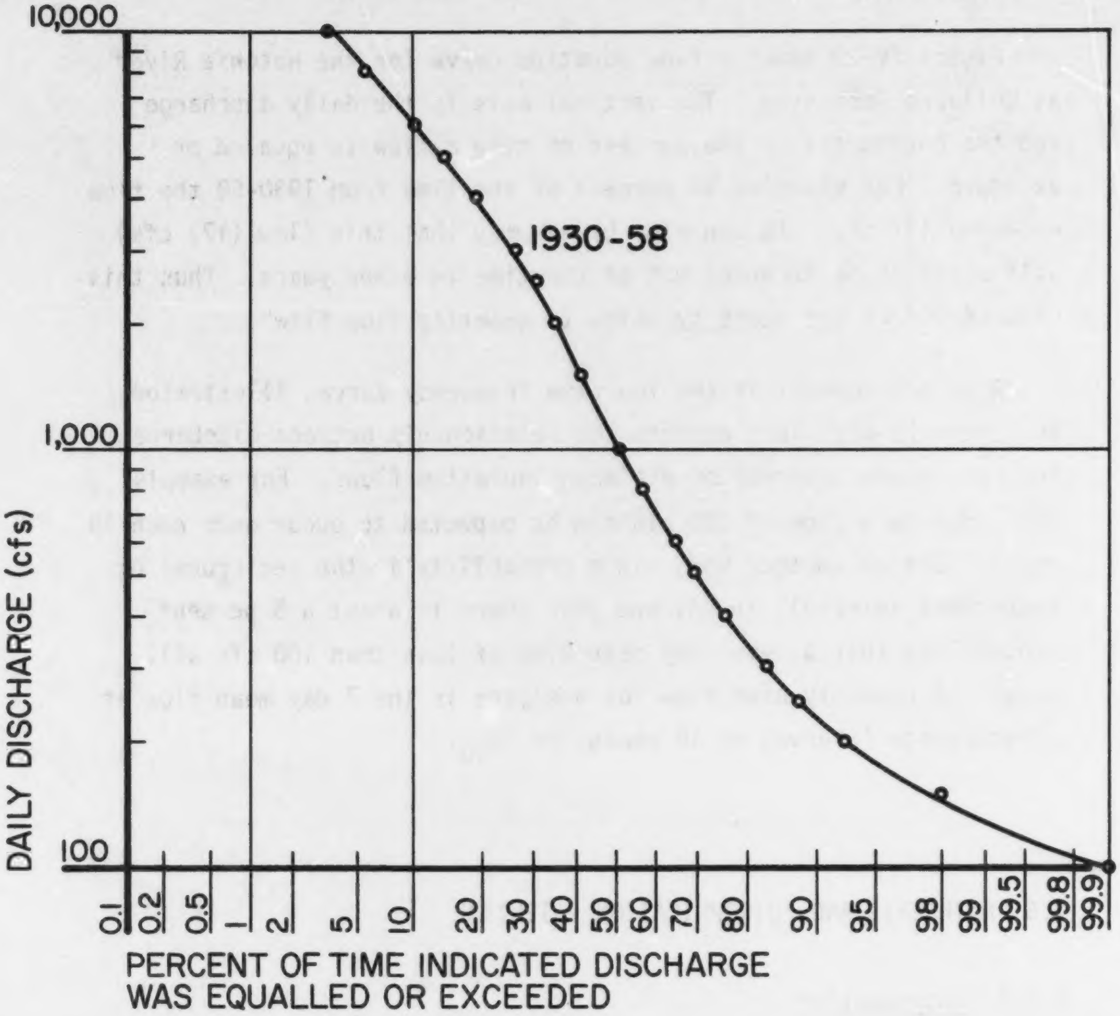

FIGURE IV-20 FLOW DURATION CURVE, HATCHIE RIVER AT BOLIVAR, TENN. (FROM CRAGWALL, 1966)

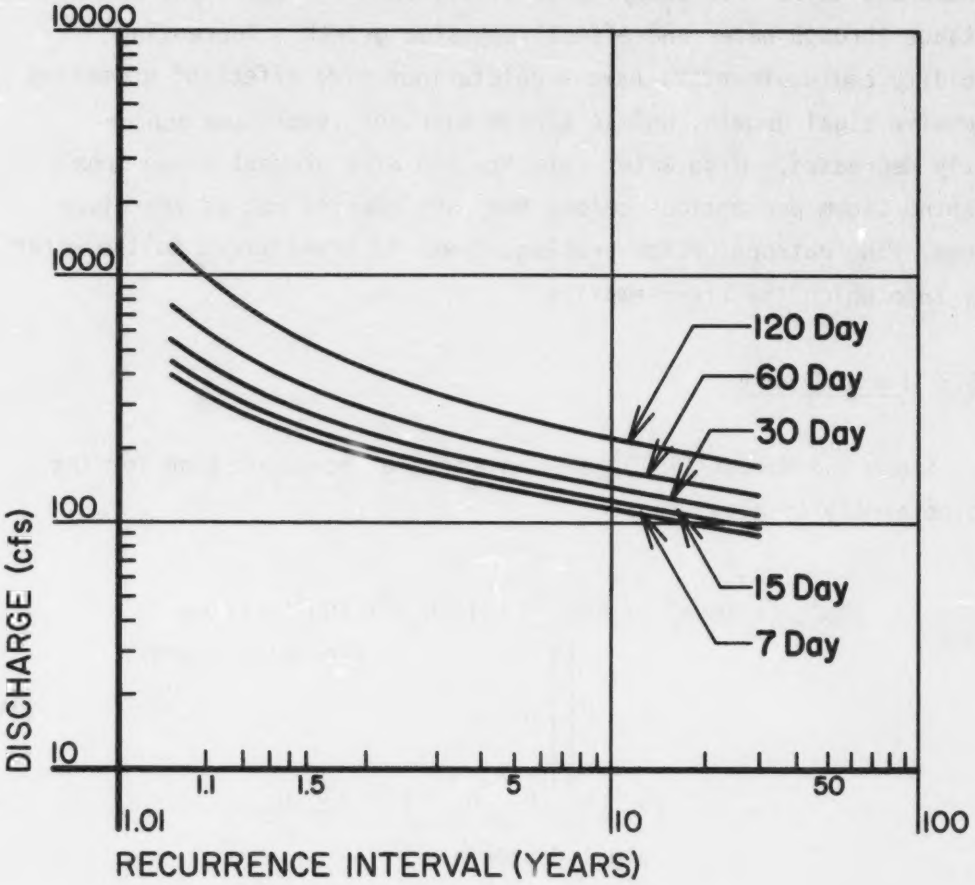

FIGURE IV-21 FREQUENCY OF LOWEST MEAN DISCHARGES OF INDICATED DURATION, HATCHIE RIVER AT BOLIVAR, TENN. (FROM CRAGWALL, 1966)

These increasing concentrations afford more favorable conditions for eutrophication, although many rivers with high nutrient levels do not have algal blooms (see Table IV-2). Algal growth can be inhibited in numerous ways. For example, turbidity can decrease light transmittance through water and effectively stop growth. Decreasing turbidity could, however, have a deleterious side effect of promoting excessive algal growth, unless stream nutrient levels are concurrently decreased. High water velocity can also prevent algae from reaching bloom proportions before they are carried out of the river system. The eutrophication problem, then, is transferred to the water body into which the river empties.

4.5.2 Basic Theory

Stumm and Morgan (1970) have proposed a representation for the stoichiometry of algal growth:

$$106CO_2 + 16NO_3^- + HPO_4^{2-} + 122H_2O + 18H^+ (\text{+ trace elements; energy}) \quad \underset{R}{\overset{P}{\rightleftharpoons}} \quad \underset{\text{algal protoplasm}}{\{C_{106}H_{263}O_{110}N_{16}P_1\}} + 138\,O_2 \qquad (IV\text{-}37)$$

where P and R represent photosynthesis and respiration, respectively. Observe that in the algal protoplasm the ratio of C:N:P is

C:N:P = 106:16:1, by atomic ratios (IV-38)

C:N:P = 41:7:1, by weight ratios (IV-39)

From the above two equations it can be inferred that only small amounts of phosphorus are needed to support algal growth in relation to the amounts of carbon and nitrogen required. If phosphorus is not present in the amount required for algal growth then algal production will be curtailed, regardless of how much of the other nutrients is available. Phosphorus is then termed growth limiting. It is possible for other elements, particularly nitrogen, and occasionally carbon or trace metals, to be growth limiting as well (Stumm and Stumm-Zollinger, 1972).

Nitrogen uptake by algae is generally in the nitrate form if nitrate is available. However, different types of fresh water algae can utilize either organic nitrogen or inorganic nitrogen in the form of ammonia, depending on what is available (Stumm and Stumm-Zollinger, 1972). Algae typically require phosphorus in an inorganic form, usually as orthophosphate ion (Kormondy, 1969).

Some indication of whether nitrogen or phosphorus is growth limiting may be made by determining the weight ratio of the appropriate forms of nitrogen and phosphorus found in a river, and comparing that with the stoichiometric ratio required for growth. This gives an idea regarding the nutrient on which control efforts should focus. Specifically, let

$$R = \frac{[TN]}{[OPO_4\text{-}P]} \qquad \text{(IV-40)}$$

where

$[TN]$ = concentration of total nitrogen in river, mg-N/l

$[OPO_4\text{-}P]$ = concentration of orthophosphate, mg-P/l

If $R>10$, phosphorus is more likely to limit than N.

If $R<5$, nitrogen is more likely limiting than P.

If $5<R<10$, a determination can not be made.

Since the N:P ratio in algal biomass can vary from species to species, this makes the determination of the limiting nutrient somewhat uncertain, and leads to the indeterminate range of $5<R<10$. If local data include an inventory of algal species present, then the N:P ratio of the known species should be used in lieu of Equation IV-39. Both Lehman, et al. (1975) and Lund (1965) provide specific algal data as well as further discussions.

The following table (Table IV-21) shows an approximate relationship between total nitrogen and total phosphorus concentrations and the potential algal biomass that can result. Both nitrogen and phosphorus must be present in the amounts shown for the resultant growth to occur.

TABLE IV-21

EUTROPHICATION POTENTIAL AS A
FUNCTION OF NUTRIENT CONCENTRATIONS

P (mg-P/1)	N (mg-N/1)	Dry Algal Cells (mg/1)	Significance
0.013	0.092	1.45	Problem threshold
0.13	0.92	14.5	Problem likely to exist
1.3	9.2	145.0	Severe problems possible

4.5.3 Estimating Instream Nutrient Concentrations

Because of the transformations that occur among the different nitrogen and phosphorus compounds it is not possible to conveniently track any particular form of nitrogen or phosphorus through a stretch of river. However, if total nitrogen and total phosphorus can be considered conservative, a mass balance approach can be easily formulated for these constituents. In reality this assumption may not be met for a variety of reasons.

For example, algae utilize nutrients, die, and settle to the bottom. Although there is a recycling of algal cell-bound nutrients, the settling rate may surpass the rate of recycling. Assuming total nitrogen and total phosphorus to be conservative should give an estimate of the upper limit of the instream concentrations of these nutrients.

The instream concentration of total nitrogen (TN) or total phosphorus (TP) resulting from a point discharge is (formulas will be presented for TN only; those for TP are exactly analogous):

$$TN_o = \frac{TN_u Q_u + TN_w Q_w}{Q_u + Q_w} \qquad \text{(IV-41A)}$$

or

$$TN_o = \frac{TN_u Q_u + w_p}{Q_u + Q_w} \qquad \text{(IV-41B)}$$

where

TN_u = instream TN upstream of discharge, mg-N/l

TN_w = concentration of TN in point discharge, mg-N/l

Q_u = flow in river upstream of point discharge, cfs

Q_w = flow rate of point discharge, cfs

TN_o = resulting instream TN concentration, mg-N/l

w_p = loading rate of point source, lb/day

The expression for TN_o is given by either Equation IV-41A or IV-41B. The appropriate form to use will depend on the form of the available data.

To determine the instream concentration of total nitrogen due to a distributed discharge, use:

$$TN = TN_o + \frac{\Delta_Q x}{Q} (TN_r - TN_o) \qquad \text{(IV-42A)}$$

or

$$TN = \frac{TN_o Q_o}{Q} + \frac{wx}{Q} \qquad \text{(IV-42B)}$$

where

TN_r = TN entering with the distributed flow, mg-N/l

TN_o = instream TN at $x = 0$, mg-N/l

x = distance downstream from the point source discharge

Q = stream flow rate at x, cfs

Q_o = stream flow rate at $x = 0$, cfs

Δ_Q = incremental flow increase per unit distance, cfs/mile

w = mass flux of TN entering the stream through the distributed source, lb/day/mile

In general, Equation IV-42B will probably be the more useful of the above two equations, since the influx of TN is expressed in terms of a total influx per unit distance. The expressions (McElroy, et al., 1976) utilized in Chapter III for TN and TP loadings can be used directly in these expressions by dividing the loading by the number of stream miles over which it occurs. The flows Q_o and Q should reflect average flows over the period the analysis is designed to cover.

The Environmental Protection Agency (1976) has performed studies relating land use to stream nutrient levels. The geographic area for which the data were collected encompassed most of the United States east of the Mississippi River. As a result of these studies, expressions were developed that related land use and stream nutrient concentrations. The expressions can aid the planner in obtaining estimates of nutrient concentrations. Data pertinent to the system under investigation should be preferentially used when available, however.

The predictive equation for total phosphorus is given by:

$$\text{Log}_{10} (\text{TP} =-1.831 + 0.0093 (\%\ \text{agric.} + \%\ \text{urban}) \qquad (\text{IV-43A})$$

where

% agric. = percent of land used for agricultural purposes
% urban = percent of land used for urban activities

Experience by the Environmental Protection Agency has shown this expression to be more appropriate when agriculture and/or forest comprise the predominant land use type(s) (even though forested land does not appear in Equation IV-43A). This equation is based on regression analysis. Accordingly, it does not describe physical mechanisms exactly. Hence there is a difference between observed values and those predicted by the equation. Table IV-22 illustrates this variability. For example, in streams draining areas with a combined agriculture plus urban land use percentage of 50%, mean

TABLE IV-22

PREDICTED MEAN TOTAL PHOSPHORUS (TP)
CONCENTRATIONS (mg/l) (FROM EPA, 1976)

% Ag + % Urb	Avg. TP	67% Limits	95% Limits
0	0.015	0.008-0.027	0.004-0.050
25	0.025	0.014-0.046	0.007-0.086
50	0.043	0.023-0.079	0.013-0.146
75	0.074	0.040-0.135	0.022-0.249
100	0.126	0.068-0.231	0.037-0.427

total phosphorus concentrations average 0.043 mg-P/l. However because of data variability, there is a 67% probability that the true value is between 0.023-0.079 mg-P/l, and a 95% probability the true value is between 0.013-0.146 mg-P/l.

For orthophosphate the regression equation is:

$$\text{Log}_{10}\ (\text{OPCONC}) + -2.208 + 0.0089\ (\%\ \text{agric.} + \text{urban}) \qquad (\text{IV-43B})$$

The coefficient of linear correlation (r) is 0.70.

The same type of predictive equation for total nitrogen has been formulated. It is expressed as:

$$\text{Log}_{10}\ (\text{TN}) = -0.278 + 0.0088\ (\%\ \text{agric.} + \%\ \text{urban}) \qquad (\text{IV-44})$$

Table IV-23 shows the predicted concentrations for different percentages of agricultural and urban coverage.

TABLE IV-23

PREDICTED TOTAL NITROGEN (TN)
CONCENTRATIONS (mg/l) (FROM EPA, 1976)

% Ag + % Urb	Avg. TN	67% Limits	95% Limits
0	0.53	0.35-0.80	0.24-1.19
25	0.87	0.58-1.31	0.39-1.96
50	1.45	0.97-2.18	0.64-3.26
75	2.41	1.61-3.62	1.07-5.42
100	4.00	2.67-6.00	1.78-9.00

4.5.4 Nutrient Accounting System

It may be desirable to determine the impact of each nutrient source on the total instream concentration in order to distinguish among the major sources. An accounting procedure utilizing Equations IV-41 and IV-42 can be developed to do this. The following steps outline the procedure.

1. Segment River. Divide the river into major segments. These segment divisions may reflect waste loading distributions or another convenient division scheme chosen at the discretion of the planner. The segments are not necessarily the same as the reaches that have previously been discussed (see Section 4.1). The delineation of reaches as described earlier is based upon lengths of river having uniform hydraulic conditions. Segments, as used here, are purely a convenience subdivision of the river. If desired, furthermore, each segment can again be subdivided when determining instream nutrient concentrations.

2. Quantify and Locate Sources of Nutrients. The quantification of point, non-point, and natural sources on the mainstem and tributaries should be accomplished using the best available data. Tabulation can be performed for each different season to reflect the discharge pattern characteristic of each season. The quantification will include total nitrogen and total phosphorus. Tabulate data in terms of average daily input (lb/day). Characterize the location of the nutrient sources by river mile. For non-point sources characterize by river mile at both the beginning and end of the source.

3. Perform Mass-Balance. Sum the known sources to determine the total nutrient loading to each segment. Then make the following comparisons:

   a. Compare the total loading with the nutrient input from the mainstem at the upstream end of the segment. This direct comparison permits an assessment of the collective impact of the nutrient sources entering a segment and the upstream contribution of the mainstem.

   b. Perform an intersource comparison to ascertain the relative impact of each nutrient source. Express the results for each source as a percent of the total loading.

When a tributary has a high percent contribution steps 1 through 3 can be repeated for the tributary itself to track the sources of the nutrients. Depending on its length the entire tributary may be considered as a single segment.

Apply Equations IV-41 and IV-42 to each reach within the segment to determine the instream nutrient concentration throughout the segment. Once this is done this step can be repeated for the next reach.

By applying this analysis one can determine the relative impact of any discharge, determined jointly by the flux of the nutrient and the discharge location.

EXAMPLE IV-9

Computing Total Nitrogen Distribution

This example illustrates the use of Equations IV-41B and IV-42B in calculating the total nitrogen distribution in a river. Suppose the user has been able to estimate the point and non-point loading of total nitrogen in a river as shown in Table IV-24.

TABLE IV-24

TOTAL NITROGEN DISTRIBUTION IN A RIVER IN RESPONSE TO POINT AND NON-POINT SOURCE LOADING

Reach Number	River Mile-Point	TN Added* (lbs/day)	TN Cummulative (lbs/day)	Q Cummulative (cfs)	TN Contration (mg-N/l)
1	0	400 L	400	300	0.25
	9.99	500 D	900	400	0.42
2	10.0	0	900	400	0.42
	14.99	700 D	1,600	600	0.50
3	15.0	800 L	2,400	700	0.64
	20.99	650 D	3,050	900	0.62
4	21.0	0	3,050	900	0.62
	26.0	900 D	3,950	1,000	0.73

*"L" indicates a localized or point source. "D" indicates a diffuse or non-point source whose range of input is over the entire reach.

If these loading rates are estimated over a year, then the flow rates used should also be average annual flows. To compute the concentration at mile 0, Equation IV-41B can be used:

$$TN_o = \frac{(0)(Q_u) + \frac{400}{8.34}}{\frac{300}{1.55}} = 0.25 \text{ mg-N/l}$$

where the following conversions were used:

$$1 \text{ MGD} = 1.55 \text{ cfs}$$

$$1 \text{ mg/l} = 8.34 \text{ lb/MG}$$

To determine the concentration at milepoint 9.99, use Equation IV-42B:

$$TN = (0.25)\,\frac{300}{400} + \frac{\dfrac{500}{8.34}}{\dfrac{400}{1.55}} = 0.42 \text{ mg-N/l}$$

Note that wx in Equation IV-42B is the 500 lb/day shown in Table IV-24. By reapplying these two basic equations for each reach the user can work downstream through the four reaches. Also note that the total nitrogen concentration has decreased slightly through reach 3, even though more TN has been added. This is because the incoming flow has served to lower the concentration by dilution.

END OF EXAMPLE IV-9

4.6 TOTAL COLIFORM BACTERIA

4.6.1 Introduction

Total coliform bacteria are considered an indicator of the presence of pathogenic organisms, and as such relate to the potential for public health problems. Allowable levels of total coliform bacteria in rivers vary from state to state and according to the water use description characterizing the particular river segment. For example, in Montana (Montana State Dept. of Health and Environmental Sciences, 1973) the raw water supply may not have more than an average of 50 MPN/100 ml* total coliforms if it is to be used as a potable water supply following simple disinfection. In water suitable for bathing, swimming and recreation, as well as growth and marginal propagation of salmonoid fishes, an average of 1,000 MPN/100 ml is allowable.

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*MPN means "Most Probable Number". Coliform organisms are not counted individually, but their densities are statistically determined and the results stated as MPN/100 ml.

Concentrations of total coliforms vary with the season of the year. Often the heaviest loadings occur during the summer months, but this impact is somewhat offset due to the more rapid die-off at higher temperatures. In the Willamette River (Figure IV-22), for example, the highest counts were actually observed from November through May.

Treated municipal sewage comprises a major source of coliform pollution. Urban stormwater runoff can also be significant, especially through combined sewer outflows. Rural storm water runoff transports significant fecal contamination from livestock pastures, poultry and pig feeding pens, and feedlots. Wildlife both within refuges and in the wilds can contribute as well. For guidance in the interpretation of preliminary coliform analyses, Table IV-25 can be used.

TABLE IV-25

TOTAL COLIFORM ANALYSIS (EPA, 1976)

If the Calculated Concentration is:	Probability of a Coliform Problem
Less than 100/100 ml	Improbable
Less than 1,000/100 ml	Possible
More than 1,000/100 ml	Probable
More than 10,000/100 ml	Highly Probable

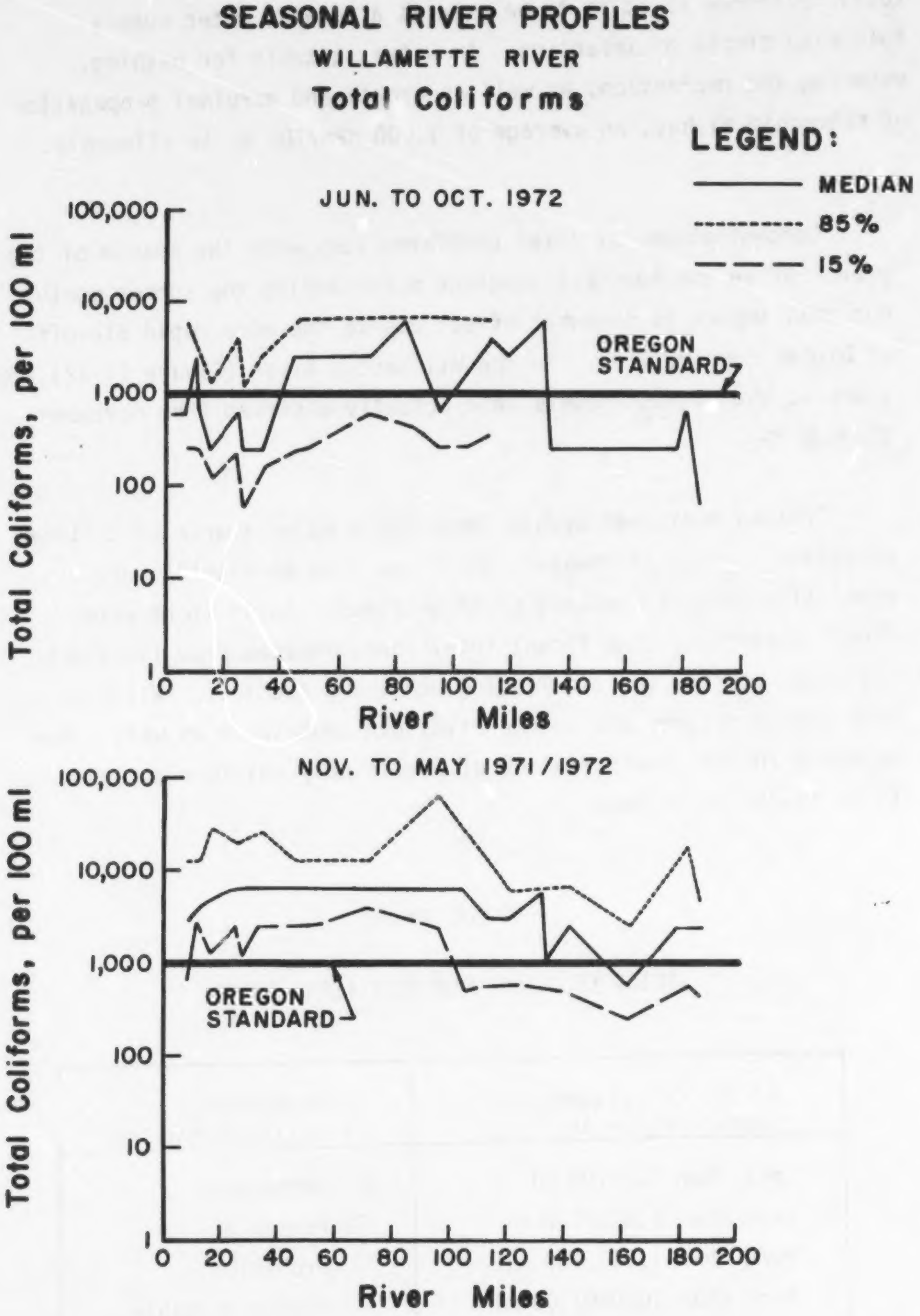


FIGURE IV-22 TOTAL COLIFORM PROFILES FOR THE WILLAMETTE RIVER (EPA, 1974)

4.6.2 Mass Balance for Total Coliforms

The mass balance equations applicable to total coliform organisms are exactly analogous to Equations IV-5, IV-7, and IV-9A and IV-9B, since first order decay is used for both. For purposes of hand computations, the following decay coefficient is acceptable:

$$k_{tc} = 1.0 + 0.02\ (T-20) \qquad \text{(IV-45)}$$

where

k_{tc} = decay coefficient for total coliforms, 1/day
T = water temperature, ^{o}C

Those equations with the widest applicability are listed below. For a point source of coliforms:

$$TC = TC_o \exp \left[\frac{-j_{tc}}{A_o} \left(A_o x + \Delta_A \frac{x^2}{2} \right) \right] \qquad \text{(IV-46)}$$

For a point and distributed source of coliforms:

$$TC = \frac{TC_r}{E_{tc}} + \left(TC_o - \frac{TC_r}{E_{tc}} \right) \left(\frac{Q_o}{Q} \right)^{E_{tc}} \qquad \text{(IV-47)}$$

For a change in coliform concentration due to a point source modification:

$$\Delta TC = \begin{cases} \Delta TC_o \exp \left[\frac{-j_{tc}}{A_o} \left(A_o x + \Delta_A \frac{x^2}{2} \right) \right] \\ \Delta TC_o \left(\frac{Q_o}{Q} \right)^{E_{tc}} \end{cases} \qquad \text{(IV-48A)}$$

where

TC = total coliform concentration, MPN/100 ml

TC_o = initial total coliform concentration, MPN/100 ml

$j_{tc} = \frac{k_{tc}}{U_o}$

TC_r = Total coliform level in distributed flow, MPN/100 ml

$E_{tc} = \frac{k_{tc}A_o + \Delta_Q}{\Delta_Q}$

Because of the potential variability in coliform loadings, seasonal analyses may be warranted. Typically the summer months are of primary concern because loadings often increase during this time period and water contact recreation is at its maximum. Major storm events may also be of interest, because of the large coliform loading that may be associated with them.

EXAMPLE IV-10

Estimating the Change in Total Coliform Levels in Response to a Waste Loading Change

Suppose the user wants to compare the change in total coliform levels, ΔTC, produced by a change ΔTC_o at a given location in a river. Further, he wants to determine how this change is affected by a distributed flow entering the river. Relevant data for the river are as follows:

U_o = 1 fps

T = 20^oC

Q_o = 500 cfs

Q_f = 800 cfs

x_L = 10 miles

k_{tc} = 1.0/day at 20°C

First the computations will be performed assuming no distributed flow. Equation IV-48A is then applicable. Computing the exponent $j_{tc}x$ (at a flow distance of 10 miles):

$$j_{tc}x = \frac{(1.0)\,(10)\,(5280)}{(24)\,(3600)\,(1)} = 0.611$$

So

$$\frac{\Delta TC}{\Delta TC_o} = \exp(-.611) = 0.54$$

or

$$\Delta TC = 0.54\ \Delta TC_o$$

For example if ΔTC_o = -1,000 MPN/100 ml then ΔTC = -540 MPN/100 ml (negative ΔTC_o indicates that the coliform level has decreased from what it previously was).

Now suppose the distributed flow of 300 cfs is included in the computation. Then,

$$E_{tc} = \frac{k_{tc}\ A_o + \Delta_Q}{\Delta_Q}$$

$$A_o = Q_o/U_o = 500/1 = 500\ \text{ft}^2$$

$$\Delta_Q = \frac{300}{10(5280)} = 0.0057\ \text{ft}^2/\text{sec}$$

$$E_{tc} = \frac{(1.0)\,(500)}{(24)\,(3600)\,(0.0057)}$$

$$= 2.02$$

Then

$$\frac{\Delta TC}{\Delta TC_o} = \left(\frac{500}{800}\right)^{2.02} = 0.39$$

or

$$\Delta TC = 0.39\ \Delta TC_o$$

For ΔTC_o = 1,000 MPN/100 ml, ΔTC = -390 MPN/100 ml.

Note that this decrease is 150 MPN/100 ml less than if no distributed flow existed.

To determine the absolute total coliform level, simply add to the original level the resulting change caused by the waste loading modification.

END OF EXAMPLE IV-10

4.7 CONSERVATIVE CONSTITUENTS

4.7.1 Introduction

Conservative constituents are those which are not reactive and remain either in solution or in suspension. They are advected through the water column at the velocity of the river with no loss of mass. The analysis of nutrients, already discussed in this report, was performed assuming they acted conservatively. Other substances, such as salinity and suspended solids (on a long term basis), can also be considered as conservative. Chapter 3 contains information on salinity in irrigation return flow for many rivers with salinity problems.

Mass Balance for Conservative Constituents

Two simple mass balance equations are sufficient for analyzing conservative constituents. The first relates the instream concentration due to a point loading:

$$S = \frac{S_u Q_u + W}{Q_u + Q_w} \qquad \text{(IV-49)}$$

where

S = resulting pollutant concentration, mg/l

S_u = upstream concentration, mg/l

Q_u = upstream flow rate, cfs

Q_w = point source flow rate, cfs

W = loading rate of pollutants, lb/day

The units of the loading rate W have to be made compatible with the other terms in Equation IV-49. This is done by using W', where

$$W' = \frac{1.55}{8.34} W$$

This relationship is obtained by using the following two conversions:

1MGD = 1.55 cfs

1mg/l = 8.34 lb/MG

When a distributed flow is present along some length of the river, then the distribution of the conservative pollutant is given by:

$$S = \frac{S_o Q_o}{Q} + \frac{wx}{Q} \qquad \text{(IV-50)}$$

where

w = distributed loading rate, lb/day/mi

x = distance downstream, miles

S_o = initial concentration (at $x = 0$), mg/l

S_o in Equation IV-50 is identical with S in Equation IV-46.

EXAMPLE IV-11

Calculating Salinity Distribution in a River

Salinity problems are receiving increased attention in the western United States, particularly relating to the economic issues in the Colorado River Basin and international impacts with Mexico. In the Colorado River Basin high salinity levels in the lower reaches adversely affect nearly twelve million people and approximately one million acres of fertile irrigated farmland (Bessler and Maletic, 1975). The salinity now averages approximately 865 mg/l at Imperial Dam and is projected to be 1,160 mg/l or more by the year 2000, unless firm control actions are taken.

As an example, suppose one is interested in determining the salinity distribution in the river shown in Figure IV-23. Assume the data shown there are averaged over a period of a year. These data, along with the salinity concentrations at different river mileposts are tabulated in Table IV-26.

To calculate S (salinity at milepoint 100) use Equation IV-49:

$$S = \frac{0.500 + (2\text{x}10^6)(1.55/8.34)}{2000} = 186 \text{ mg/l}$$

At milepost 199.9, Equation IV-50 is appropriate and S is given by:

$$S = \frac{(186)(2000)}{5000} + \frac{(4\text{x}10^6)(1.55/8.34)}{5000} = 223 \text{ mg/l}$$

TABLE IV-26

SALINITY DISTRIBUTION IN A HYPOTHETICAL RIVER

Reach Number	River Mile Point	Salinity Added* (lbs/day)		Salinity Cumulative (lbs/day)	Q Cumulative (cfs)	Salinity Concentration (mg/l)
1	0	0		0	500	0
	99.9	0		0	500	0
2	100	$2x10^6$	L	$2x10^6$	2000	186
	199.9	$4x10^6$	D	$6x10^6$	5000	223
3	200	0		$6x10^6$	5000	223
	279.9	0		$6x10^6$	5000	223
4	280	$-1.2x10^6$	L	$4.8x10^6$	4000	223
	359.9	0		$4.8x10^6$	4000	223
5	360	0		$4.8x10^6$	4000	223
	449.9	$25x10^6$	D	$29.8x10^6$	9000	615
6	450	0		$29.8x10^6$	9000	615
	499.9	0		$29.8x10^6$	9000	615
7	500	$8\ 10^6$	L	$37.8x10^6$	12000	585
	524.9	0		$37.8x10^6$	12000	585
8	525	$-7.9x10^6$	L	$29.9x10^6$	9500	585
	599.9	0		$29.9x10^6$	9500	585
9	600	$-4.7x10^6$	L	$25.2x10^6$	8000	585
	649.9	0		$25.2x10^6$	8000	585
10	650	0		$25.2x10^6$	8000	585
	750	$20x10^6$	D	$45.2x10^6$	10000	840

*'L' indicates a localized or point source at the milepoint shown in the same row.

'D' indicates a diffuse or non-point source ending at the milepoint shown in the same row and beginning at the milepoint in the above row.

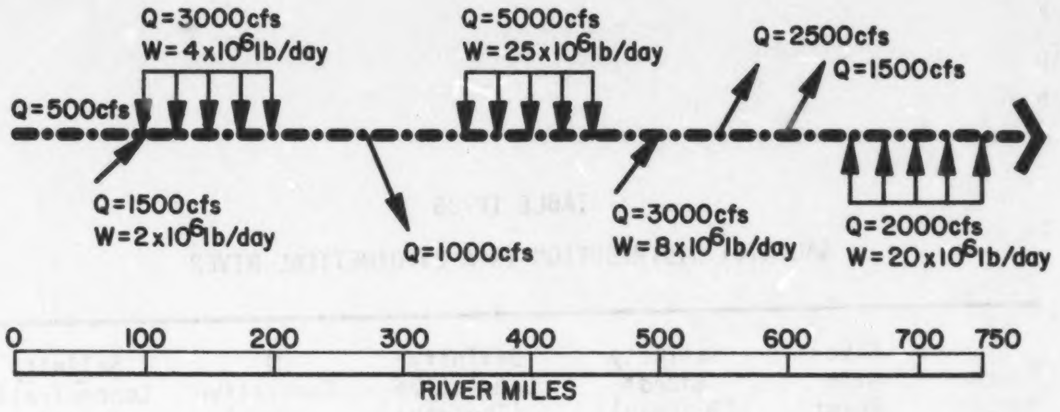

Q=3000cfs
W=4x10⁶lb/day
Q=5000cfs
W=25x10⁶lb/day
Q=2500cfs
Q=1500cfs
Q=500cfs
Q=1500cfs
W=2x10⁶lb/day
Q=1000cfs
Q=3000cfs
W=8x10⁶lb/day
Q=2000cfs
W=20x10⁶lb/day
0
100
200
300
400
500
600
700
750
RIVER MILES

FIGURE IV-23 SALINITY DISTRIBUTION IN A HYPOTHETICAL RIVER

At milepoint 280, 1,000 cfs of flow leaves the mainstem (perhaps for irrigation purposes). The concentration of salinity in this flow is the same as that in the mainstem. So the mass rate of withdrawal is

$$W = \frac{-8.34}{1.55} (223 \times 1000) = -1.2\text{x}10^6 \text{ lb/day}$$

A negative sign is used to signify a withdrawal. Completing the remainder of the table is solely a matter of reapplying these basic concepts.

END OF EXAMPLE IV-11

4.8 SEDIMENTATION

4.8.1 Introduction

One of the more difficult classes of hydraulic engineering problems associated with rivers involves the erosion, transportation, and deposition of sediment. Sedimentation is important economically, particularly relating to filling of reservoirs and harbors, and to maintaining channel navigability and stability. Table IV-2, located in Section 4.1, documents some suspended solids problems encountered in eight major U.S. waterways.

The sediment loading in a river can be divided into two components: the bed material load and the wash load. The bed material load is composed of those solid particles represented in the bed. The transport of this material is accomplished both along the bed (bed load) and suspended within the water (suspended load). Although there is no sharp demarcation delineating bed load from suspended load, many researchers have developed individual expressions for each transport component. The total bed material load then is the sum of the bed load and the suspended load. Other researchers have developed a unified theory from which the total bed material load can be predicted from a single expression.

The wash load is usually produced through land erosion, rather than channel scour. Wash load is composed of grain sizes finer than the bulk of the bed material. It readily remains in suspension and is washed out of the river without being deposited. A definite relationship between the hydraulic properties of a river and the wash load capacity apparently does not exist, making it difficult to advance an analytical method for washload prediction (Graf, 1971). Not all the erodible material entering a stream is transported as wash load, but a large portion may become part of the bed material and be transported as bed material load.

As a guide in evaluating whether a river is carrying a significant quantity of suspended sediment, Table IV-27 can be consulted. 100 mg/l is the delineation between a potential and probable problem. In a table previously introduced (Table IV-1), a reference level of 80 mg/l was set for protection of aquatic life.

TABLE IV-27

RELATIONSHIP OF TOTAL SUSPENDED SEDIMENT CONCENTRATION TO PROBLEM POTENTIAL (AFTER EPA, 1976)

If Calculated Concentration is:	Probability of a Problem
Less than 10 mg/l	Improbable
Less than 100 mg/l	Potential
More than 100 mg/l	Probable

4.8.2 Long-Term Sediment Loading from Runoff

The procedures outlined in Chapter III will permit an assessment of the sediment loading to a river on a long-term basis. When using those procedures care should be taken to incorporate the entire drainage area of the watershed. As an estimate, the loading can be assumed conservative (i.e. all sediment that comes into the river will be washed out of the river over an extended time period). Under that assumption the procedure outlined in Section 4.7 can be utilized for an estimate of average yearly suspended solids concentrations at locations throughout the river system. This result should be interpreted only as an indicator of the impact of the runoff on sediment loads within a river and not as actual suspended solids concentrations. Not all of the incoming sediment will be transported as suspended load since a large fraction can be transported as bed load. The transport process is generally of an intermittent nature with higher concentrations occurring during periods of high flow.

Care should be taken not to apply the conservative assumption at points on a river where that assumption is clearly violated, such as at reservoirs which can be efficient sediment traps. An example for the computation of sediment loading to rivers has been considered in Chapter 3.

4.8.3 Bed Material Load

As previously mentioned, the estimation of bed material transport poses a difficult problem, and is an area where there is no consensus regarding the best predictive relationship to use. Numerous bed material load relationships (Task Committee on Preparation of Sedimentation Manual, 1971) have been developed over the past century, some requiring considerably more input data than others. In this report the DuBoys relationship (Task Committee on Preparation of Sedimentation Manual, 1971) will be used in part because of its simplicity. The relationship, which is restricted to uniform flow in alluvial channels, is:

$$g_{st} = \Psi\tau_o (\tau_o - \tau_c) \qquad \text{(IV-51)}$$

where

g_{st} = bed material discharge, lb/sec/ft of width

Ψ = coefficient depending on grain size, ft^3/lb/sec

τ_o = γR_H S, bed shear stress, lb/ft^2

γ = specific weight of water, lb/ft^3

R_H = hydraulic radius, ft

S = slope of stream, ft/ft

τ_c = critical shear stress, lb/ft^2

The values of Ψ and τ_c can be expressed as functions of the median size (by weight) of the bed sediment (d_{50}). These relationships are expressed graphically in Figure IV-24. To aid in determining d_{50} Table I -28 is presented to show the size range of sediment and each associated class name. If the class name of the predominant sediment type comprising a stream bed is known, then the sediment size (in mm) can be estimated. Appendix C also contains d_{50} data for numerous rivers and streams.

If d_{50} can be estimated then Ψ and τ_c can easily be evaluated, leaving only τ_0 to compute. A summary of hydraulic radii (the ratio of cross-sectional area to wetted perimeter) for different channel

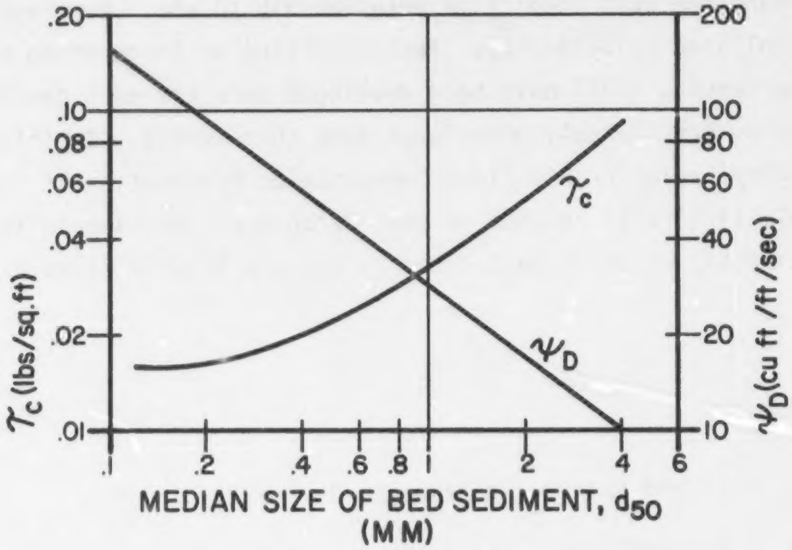


FIGURE IV-24 Ψ AND τ_c FOR DUBOYS RELATIONSHIP AS FUNCTIONS OF MEDIAN SIZE OF BED SEDIMENT (TASK COMMITTEE ON PREPARATION OF SEDIMENTATION MANUAL, 1971)

geometries is shown in Figure IV-25. For very wide, shallow channels, the hydraulic radius approximately equals the depth of flow. Many river cross-sections can be approximated by a parabolic section. To calculate "c" in the relationship for hydraulic radius of a parabolic section, refer to Table IV-29.

TABLE IV-28

SEDIMENT GRADE SCALE (TASK COMMITTEE ON PREPARATION OF SEDIMENTATION MANUAL, 1971)

	Size Range				Approximate Sieve Mesh Openings Per Inch	
Class Name	Millimeters		Microns	Inches	Tyler	United States Standard
Very large boulders		4096-2048		160-80		
Large boulders		2048-1024		80-40		
Medium boulders		1024-512		40-20		
Small boulders		512-256		20-10		
Large cobbles		246-128		10-5		
Small cobbles		128-64		5-2.5		
Very coarse gravel		64-32		2.5 -1.3		
Coarse gravel		32-16		1.3 -0.6		
Medium gravel		16-8		0.6 -0.3	2-1/2	
Fine gravel		8-4		0.3 -0.16	5	5
Very fine gravel		4-2		0.16-0.08	9	10
Very coarse sand	2-1	2.000-1.000	2000-1000		16	18
Coarse sand	1-1/2	1.000-0.500	1000-500		32	35
Medium sand	1/2-1/4	0.500-0.250	500-250		60	60
Find sand	1/4-1/8	0.250-0.125	250-125		115	120
Very fine sand	1/8-1/16	0.125-0.062	125-62		250	230
Coarse silt	1/16-1/32	0.062-0.031	62-31			
Medium silt	1/32-1/64	0.031-0.016	31-16			
Fine silt	1/64-1/128	0.016-0.008	16-8			
Very fine silt	1/128-1/256	0.008-0.004	8-4			
Coarse clay	1/256-1/512	0.004-0.0020	4-2			
Medium clay	1/512-1/024	0.0020-0.0010	2-1			
Fine clay	1/1024-1/2048	0.0010-0.0005	1-0.5			
Very fine clay	1/2048-1/4096	0.0005-0.00024	0.5-0.24			

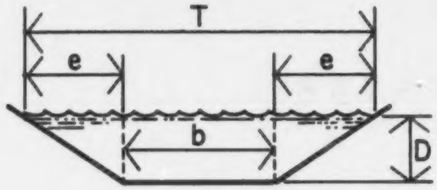
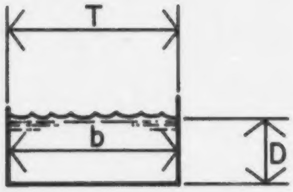
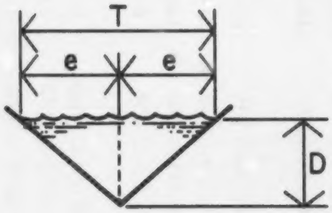
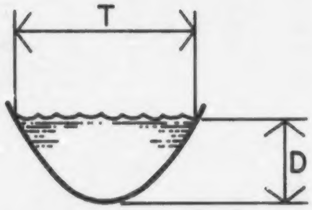
CHANNEL SLOPE	HYDRAULIC RADIUS
 Trapezoidal	$\frac{(1 + zx)\ D}{1 + 2\ x\ \sqrt{1 + x^2}}$, x = D/b, z = e/D
 Rectangular	$\frac{bD}{b + 2D}$
 Triangular	$\frac{zD}{2\ \sqrt{1 + z^2}}$ z = e/D
 Parabolic	cD (for c, see Table IV-29)

FIGURE IV-25 HYDRAULIC RADII FOR DIFFERENT CHANNEL SHAPES (FROM KING, 1954)

If the bed slope is unknown it can be estimated by using a topographic map and finding contour lines approximately five hundred feet above and below the point on the river where the measurement is to be made. Dividing this elevation difference by the horizontal distance over which the difference is measured, produces the slope.

TABLE IV-29

COMPUTING D/T FOR DETERMINING THE HYDRAULIC RADIUS OF A PARABOLIC SECTION (FROM KING, 1954)

$\frac{D}{T}$*	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0	.667	.667	.666	.665	.664	.662	.660	.658	.656	.653
.1	.650	.646	.643	.639	.635	.631	.626	.622	.617	.612
.2	.607	.602	.597	.592	.586	.581	.575	.570	.564	.559
.3	.554	.548	.543	.537	.532	.526	.521	.516	.510	.505
.4	.500	.495	.490	.485	.480	.475	.470	.465	.460	.455
.5	.451	.446	.442	.437	.433	.428	.424	.420	.416	.412
.6	.408	.404	.400	.396	.392	.388	.385	.381	.377	.374
.7	.370	.367	.364	.360	.357	.354	.351	.348	.344	.341
.8	.338	.335	.333	.330	.327	.324	.321	.319	.316	.313
.9	.311	.308	.306	.303	.301	.298	.296	.294	.291	.289

*$\frac{D}{T} = c$

Adequate methods that are within the scope of this report and which would provide a straightforward estimation of suspended sediment discharge presently do not exist. Most relationships require a known reference level concentration at some depth within the river to predict the concentration at another depth (Morris and Wiggert, 1972). To determine the suspended sediment load, then, a summation of contributions at each depth must be made. Since these formulas apply to one grain size this procedure should be repeated for all grain sizes present. Einstein (Graf, 1971) has developed a method for computing suspended sediment discharge that does not require knowledge of a reference concentration, but it is an advanced approach. For this report the contribution of the suspended load will be estimated from the bed material load by the relationship given in Table IV-30. The relationship in Table IV-30 is valid for graded channels (by graded is meant that the slope is stable over time, being neither steepened nor flattened by flow or other influence).

TABLE IV-30

RELATIONSHIP BETWEEN WIDTH TO DEPTH RATIO OF A GRADED STREAM AND THE SUSPENDED AND BED LOAD DISCHARGE (AFTER FENWICK, 1969)

Suspended Load % of Total Bed Material Load	Bed Load % of Total Bed Material Load	Width-Depth Ratio	M %
85-100	0-15	7	30-100
65-85	15-35	7-25	0-30
30-65	35-70	25	0

There is a relationship between M (a weighted mean percent of the fine sediments in the bed and bank) and the width to depth ratio for graded channels (Schumm, 1960). (See Section 4.8.4 for a detailed discussion). Once the width to depth ratio for the stream in question is determined, the suspended load can then be approximated after first computing the bed material load, and then using Table IV-30.

Once the suspended load discharge is estimated the average concentration at a section can be computed by:

$$C_{ss} = \frac{G_{ss}}{Q} 1.6 \times 10^4 \qquad \text{(IV-52A)}$$

or

$$C_{ss} = \frac{g_{ss}}{q} 1.6 \times 10^4 \qquad \text{(IV-52B)}$$

where

C_{ss} = average suspended solids concentration, mg/l

G_{ss} = suspended solids discharge, lb/sec

Q = flow rate, cfs

g_{ss} = suspended solids discharge per unit width, lb/sec/ft

q = flow rate per unit width, cfs/ft

4.8.4 Alluvial Channel Stability

Alluvial channel beds are not always graded, or in a state of equilibrium. Physical modification of the stream, such as building an upstream reservoir, or altering watershed characteristics can cause a graded stream to depart from equilibrium. The stream will then have a tendency to either erode the bed, or to deposit some of the sediment being transported. These two conditions are known, respectively, as degradation and aggradation. Schumm (1960) developed a relationship based on data from alluvial channels that enables one to estimate whether a stream is aggrading or degrading based on the presence of silt-clay, designated M, composing the perimeter of a channel. For a graded channel:

$$W/D = 255\ M^{-1.08} \qquad \text{(IV-53)}$$

where

$$M = \frac{S_c \times W + S_b \times 2D}{W + 2D}$$

S_c = percent of silt and clay in channel alluvium

S_b = percent of silt and clay in bank alluvium

D = channel depth

W = channel width

Equation IV-53 is the same relationship as expressed in Table IV-30 between W/D and M. Schumm (1960) chose silt and clay to be those sediments with sizes less than 0.074 mm (200-mesh sieve). If a channel is aggrading, then the W/D ratio is actually larger than predicted by Equation IV-53. If it is degrading, the W/D ratio will be smaller than expected. Appendix C contains M-values for sixty-nine graded channel sections.

EXAMPLE IV-12

Estimation of Bed Material Load

Table IV-31 shows characteristics of the Colorado River at Taylor's Ferry, California, and of the Niobrara River near Cody, Nebraska. Suppose one desires to calculate the bed load for the Colorado River at this location for flow ranges of 8-35 cfs/ft. The following data will be used:

d_{50} = 0.33 mm

γ = 62.4 lb/ft^3, at 60°F

S = 0.000217 ft/ft

TABLE IV-31

CHARACTERISTICS OF THE COLORADO AND NIOBRARA RIVERS (TASK COMMITTEE ON PREPARATION OF SEDIMENTATION MANUAL, 1971)

Data	Stream	
	Colorado River (Taylor's Ferry)	Niobrara River (Cody, Neb.)
Depth range, ft	4-12	0.7-1.3
Range in q, in cubic feet per second per foot of width	8-35	1.7-5
Mean width, in feet	350	110
Slope, in feet per foot		
Minimum value	0.000147	0.00116
Maximum value	0.000333	0.00126
Value used in calculations	0.000217	0.00129
Water temperature, in degrees Fahrenheit		
Minimum value	48	33
Maximum value	81	86
Value used in calculations	60	60
Geometric mean*sediment size, in millimeters	0.320	0.283
d_{35}, in millimeters	0.287	0.233
d_{50}, in millimeters	0.330	0.277
d_{65}, in millimeters	0.378	0.335
d_{90}, in millimeters	0.530	0.530
Mean size, d_m, in millimeters	0.396	0.342

*The geometric mean of a set of values X_n is $\left(\prod_{i=1}^{n} X_n\right)^{1/n}$. Thus the geometric mean of the values 1, 2, 3, and 4 is $(1 \times 2 \times 3 \times 4)^{1/4} = 2.213$. Compare with arithmetic mean of 2.5.

Using Figure IV-24 one finds

$$\Psi = 64$$

$$\tau_c = 0.019$$

All that remains is the computation of the hydraulic radius. Since the width is much greater than the depth, you can assume $R_H = D$.

$$R_H = \begin{cases} 4 \text{ ft at } q = 8 \text{ cfs/ft} \\ 12 \text{ ft at } q = 35 \text{ cfs/ft} \end{cases}$$

Using Equation IV-51 it is found that the bed material load is

$$g_{st} = \begin{cases} 0.12 \text{ lb/sec/ft at } q = 8 \text{ cfs/ft} \\ 1.5 \text{ lb/sec/ft at } q = 35 \text{ cfs/ft} \end{cases}$$

The actual bed material load observed at Taylor's Ferry has been compared with the DuBoys prediction for a range of flow rates (Task Committee on Preparation of Sedimentation Manual, 1971). This relationship is shown in Figure IV-26. (The DuBoys curve in Figure IV-26 does not quite match the calculations in this example because slightly different data were used). Observe that the DuBoys relationship overpredicts the bed material load for nearly all flow ranges. This pattern is repeated for the Niobrara River (Figure IV-27). This suggests that the bed material load estimated by the DuBoys relationship will in general exceed the actual bed material load. This is further substantiated by other work (Stall, et al, 1958). The more accurate predictions of bed material load occur under high flow conditions, which is generally when the prediction of bed material load is most important.

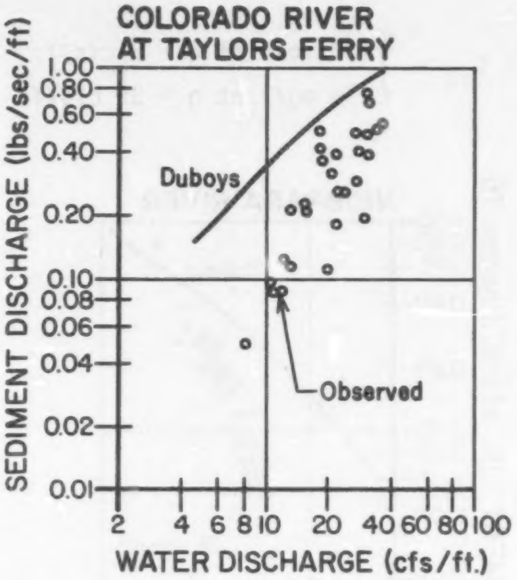


FIGURE IV-26 SEDIMENT DISCHARGE AS A FUNCTION OF WATER DISCHARGE FOR THE COLORADO RIVER AT TAYLOR'S FERRY (TASK COMMITTEE ON PREPARATION OF SEDIMENTATION MANUAL, 1971)

To estimate the suspended load contribution first calculate the width-depth ratio:

$$WD = \begin{cases} 88 \text{ at } q = 8 \text{ cfs/ft} \\ 29 \text{ at } q = 35 \text{ cfs/ft} \end{cases}$$

In both cases W/D >25. Referring to Table IV-30, the suspended load should be between 30 and 65 percent of the bed material load. Assume it is on the lower end of the scale, about 40%. Then the suspended load is

$$g_{ss} = \begin{cases} 0.05 \text{ lb/sec/ft at } q = 8 \text{ cfs/ft} \\ 0.60 \text{ lb/sec/ft at } q = 35 \text{ cfs/ft} \end{cases}$$

or

$$C_{ss} = \begin{cases} 100 \text{ mg/l at } q = 8 \text{ cfs/ft} \\ 275 \text{ mg/l at } q = 35 \text{ cfs/ft} \end{cases}$$

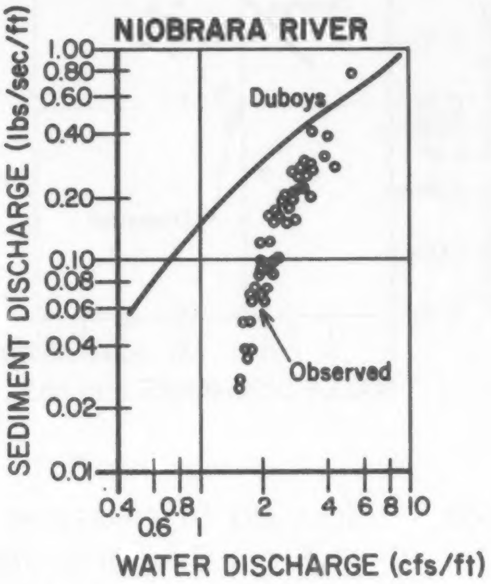


FIGURE IV-27 SEDIMENT DISCHARGE AS A FUNCTION OF WATER DISCHARGE FOR THE NIOBRARA RIVER AT CODY, NEBRASKA (TASK COMMITTEE ON PREPARATION OF SEDIMENTATION MANUAL, 1971)

from Equation IV-52B. These concentrations indicate that suspended sediment concentrations are excessively high throughout the range of flows normally encountered at Taylor's Ferry. Data on suspended sediment concentrations have been gathered at Taylor's Ferry (U.S.

Bureau of Reclamation, 1958). The average of 30 measurements taken there are as follows:

Q = 7350 cfs (or q = 21 cfs/ft)

C_{ss} = 132 mg/l

Observed range of suspended sediment
concentration: 40-277 mg/l

These values are generally within the range predicted in this example.

——————————— END OF EXAMPLE IV-12 ———————————

4.9 PRESENTATION OF RESULTS

In essence, the purpose of this chapter has been to predict instream levels of various pollutants and water quality indices for the purpose of identifying potential problem areas. The planner must be concerned not only with the location of problem areas but the time of year they occur as well. This is the case because as Table IV-6 has shown, the impact of pollutants is seasonally influenced. Further, because of the comprehensive nature of the 1972 Water Pollution Control Act Amendments, planners can not concentrate on one pollutant to the exclusion of others, but rather, should methodically proceed to determine the impact of a wide spectrum of pollutants.

As the analyses are done, a display of results becomes critical if interpretation of findings is to be done rapidly and yet accurately. A graphical presentation is often a very meaningful method of displaying results. For example, suppose a dissolved oxygen analysis is to be performed. The planner may want to answer the following questions:

- At what points do the instream DO concentrations fall below the standard (for a selected loading scheme)?

- What appears to be the major cause of the low levels?

- How do these levels relate to saturation?

- How do the DO levels respond to different loading schemes?

The answers to these and similar questions can be shown by graphs, such as those which might be developed from Example IV-5 (see Figure IV-28). The only points plotted in the figure are those tabulated in the example. These are joined by straight lines. Actually, curved lines should result, but this would have required finding DO values at additional milepoints.

Only on the downstream end of the river reach studied, does the DO level fall below the 5.0 mg/l standard used. The cause of these fairly low DO levels is directly related to the large waste discharge at mile 12 on the river. Just above that point, the DO level is seen to be increasing toward saturation.

It should be recalled from Example IV-5 that the DO would still be decreasing downstream of milepoint 16 if there were no discharge coming into the river at that point. Even though there is, it appears certain that a further DO decrease will occur proceeding downstream, and which would warrant further investigation of that reach.

Two important questions regarding low DO levels are resolved by this graphical representation. These questions are: where do low DO levels occur and what causes them? Also, by displaying the locations and magnitudes of the waste inputs, an assessment can be made of the relative magnitude of each. Although only point sources are shown in Figure IV-28, nonpoint sources, such as those considered in Example IV-11 can also be readily illustrated.

Results of alternative waste loading schemes and other stream flow conditions can similarly be presented, and such display could supply an

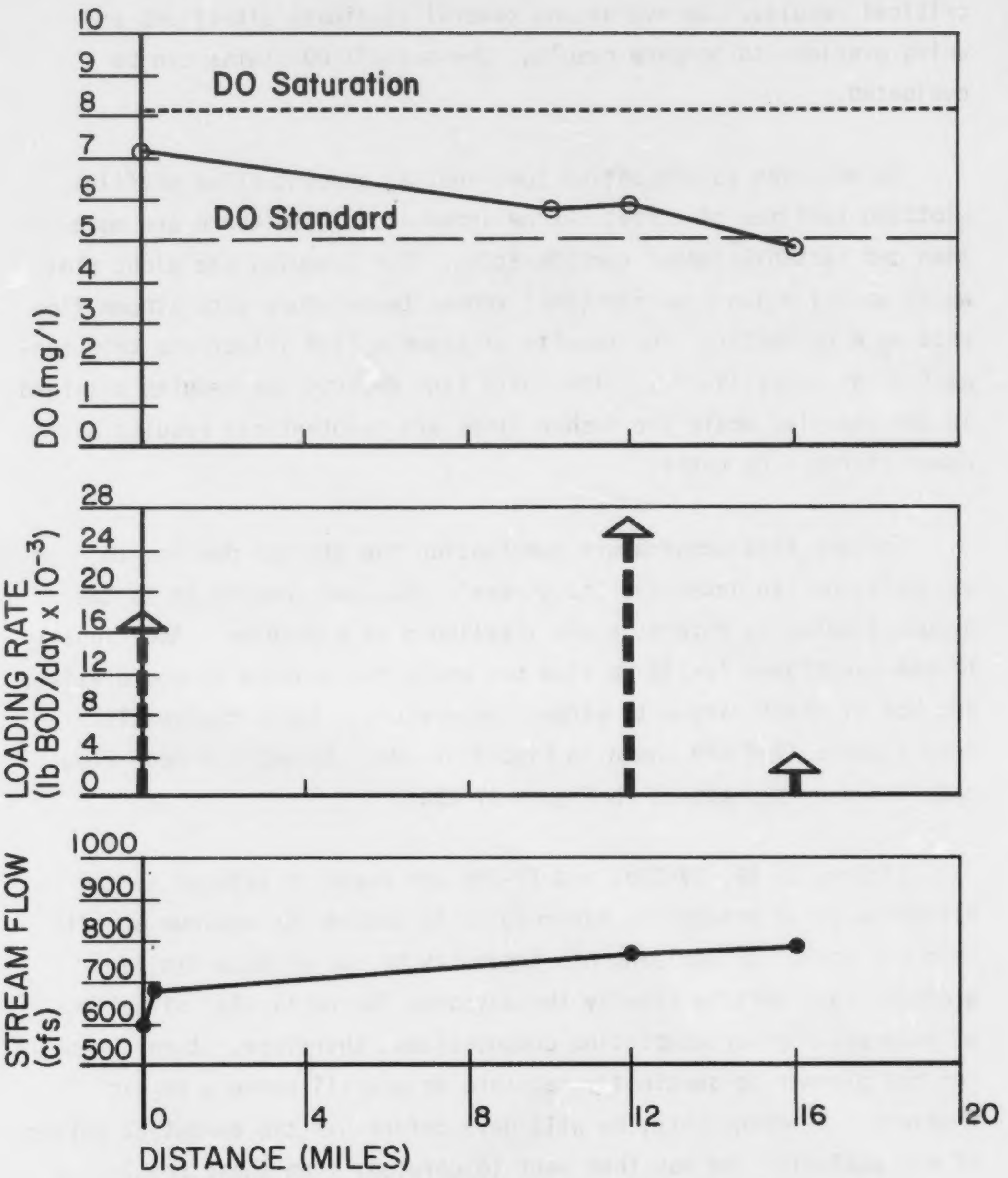


FIGURE IV-28 GRAPHICAL PRESENTATION OF RESULTS OF EXAMPLE IV-5

answer to the question of potential impact on DO. It may not be apparent, for example, which hydrologic conditions produce the most critical results. By evaluating several candidate situations and using overlays to compare results, the overall DO status can be evaluated.

In addition to presenting longitudinal concentration profiles, plotting families of curves can be informative when there are more than two variables under consideration. For example, one might plot waste assimilation capacity (WAC) versus temperature with stream flow rate as a parameter. The results of Example IV-4 illustrate this concept (see Figure IV-29a). The solid line depicts the results obtained in the example, while the dashed lines are hypothetical results at other stream flow rates.

For any flow-temperature combination the planner desires to evaluate, he can determine the stream's WAC, and compare it to the actual loading to determine the likelihood of a problem. Additionally he can use Figure IV-29b to find out where the minimum DO would exist, and how it might vary with stream temperature. (Only the results from Example IV-4 are shown in Figure IV-29b. Curves for each flow rate could be plotted as in Figure IV-29a).

Figures IV-28, IV-29a, and IV-29b are meant to suggest to the planner ways of presenting his results to obtain the maximum benefit from his work. By applying his ingenuity he can produce similar graphics that portray clearly the outcomes for particular situations of interest. After completing computations, therefore, it only remains for the planner to succinctly tabulate an overall summary of his findings. By doing this, he will have before him the essential points of his analysis. He may then want to develop, from these results, a priority system defining an order of further investigation. An example of how such a table of results might look is Table IV-32. Although a table can only contain the most germane information, supplementing it with a more detailed description of problem areas

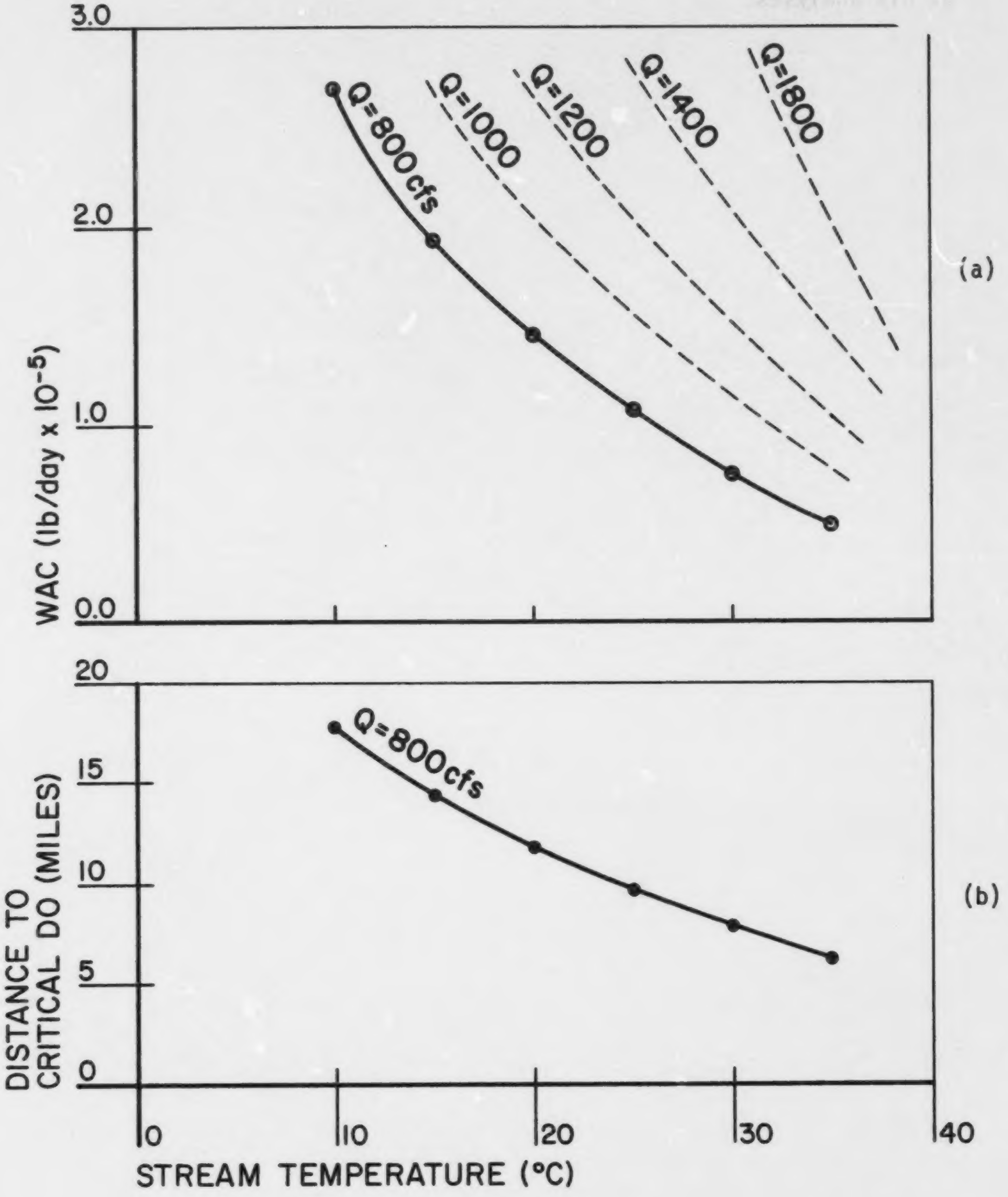


FIGURE IV-29 GRAPHICAL RESULTS OF EXAMPLE IV-4

and incorporating graphics should give the planner fingertip recall of his analyses.

TABLE IV-32

SUMMARY OF PRELIMINARY WATER QUALITY ANALYSIS OF THE HYPOTHETICAL RAPPAHAN RIVER FROM RIVERMILE 280 TO RIVERMILE 40

Water Quality Parameter	Waste Loading Conditions	Hydrologic Conditions	Waste Loads Neglected	Estimated Critical Values	Location (Rivermile)	Probable Cause	Comments
Dissolved Oxygen	Present Loading of CBOD and NBOD from point sources	August low flow (7 day duration, 10 year recurrence)	All nonpoint sources	4.1-4.9 mg/l	260 - 240	Frederick STP	
	Same as Above	Same as Above	Same as Above	4.5-5.0 mg/l	70 - 50	Saluda STP and Ferguson's Cannery	
	BPT of all point sources	Same as Above	Same as Above	None			All DO values exceed 5.5 mg/l
Temperature	No significant thermal loads, other than natural	August low flow (7 day duration, 10 year recurrence)		80°F	270 - 250	Meteorology	80°F is acceptable
	Removal of vegetation in upper reaches (urbanization)	Same as Above		91°F	270 - 250	Additional solar radiation	Potential increase significant
Nutrients	Present distribution of point sources, agricultural sources, and duck farm	Spring low flow	All major known sources considered	0.1-0.8 mg/P/l 0.3-1.5 mg/N/l	240 - 40	Agriculture and duck farms	High concentrations throughout system
	BPT and dry duck farms	Same as Above	Same as Above	0.1-0.3 mg/P/l 0.1-0.6 mg-N/l	240 - 40	Agriculture	Runoff from agricultural lands significant
Total Coliforms	Present distribution of point sources and duck farms	August low flow (7 day duration, 10 year recurrence)	Agricultural & one feedlot	10,000 MPN/100ml	240 - 200	Duck Farms	Consider analyzing below mile 40
	BPT and dry duck farms	Same as Above	Same as Above	10,000 MNP/100ml			
Conservative Constituents	No analysis yet See Comments						Perform salinity analysis when more data available

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CHAPTER 5

IMPOUNDMENTS

5.1 INTRODUCTION

This chapter contains several methods for assessing water quality and physical conditions in impoundments. The general topics covered are sediment accumulation, thermal stratification, DO-BOD, and eutrophication. The methods developed are easy to use and require readily obtainable data. Because the methods depend upon a number of simplifying assumptions, estimates should be taken only as a guide pending further analysis.

Some of the techniques are more mechanistic and reliable than others. For example, the thermal stratification technique is based upon output of a calibrated and validated hydrothermal model. The model has been shown to be a good one, and to the extent that physical conditions in the studied impoundments resemble those of the model, results should be very reliable. On the other hand, the methods for predicting eutrophication are empirical and based upon correlations between historical water quality conditions and algal productivity in a number of lakes and reservoirs. Because algal blooms are sensitive to the presence of toxicants and factors other than those involved in the estimation methods, the methods for predicting eutrophication will occasionally be inapplicable. Since the planner may not be able to assess applicability in specific cases, results may occasionally be inaccurate.

In using the techniques to be presented, it is important to apply good "engineering judgement" particularly where sequential application of methods is likely to result in cumulative errors. Such would be the case, for example, in evaluating impoundment hypolimnion DO problems resulting from algal blooms. If methods presented below are used to evaluate hypolimnion DO, the planner should determine when stratification occurs, the magnitude of point and non-point source BOD loads, and algal productivity and settling rates. From all of this, he may then predict BOD and DO levels in the hypolimnion. Since each of these techniques has an error associated with it, the end result of the computation will have a significant error envelope and results must be interpreted accordingly. The best way to use any of the techniques is to assume a range of values for input parameters in order to obtain a range of results within which the studied impoundment is likely to fall.

The methods presented below are arranged in an order such that the planner should be able to use each if he has read preceding materials. The order of presentation is:

- Impoundment stratification
- Sediment accumulation
- Eutrophication
- Impoundment dissolved oxygen

It is strongly recommended that all materials presented be read prior to applying any of the methods. In this way a better perspective can be obtained on the kinds of problems covered and what can be done using hand calculations.

5.2 IMPOUNDMENT STRATIFICATION

5.2.1 Discussion

The density of water is strongly influenced by temperature and by the concentration of dissolved and suspended matter. Figure V-1 shows densities for water as a function of temperature and dissolved solids concentration (from Chen and Orlob, 1973).

Regardless of the reason for density differences, water of lowest density tends to move upward and reside on the surface of an impoundment while water of greatest density tends to sink. Inflowing water seeks an impoundment level containing water of the same density. Figure V-2 shows this effect schematically.

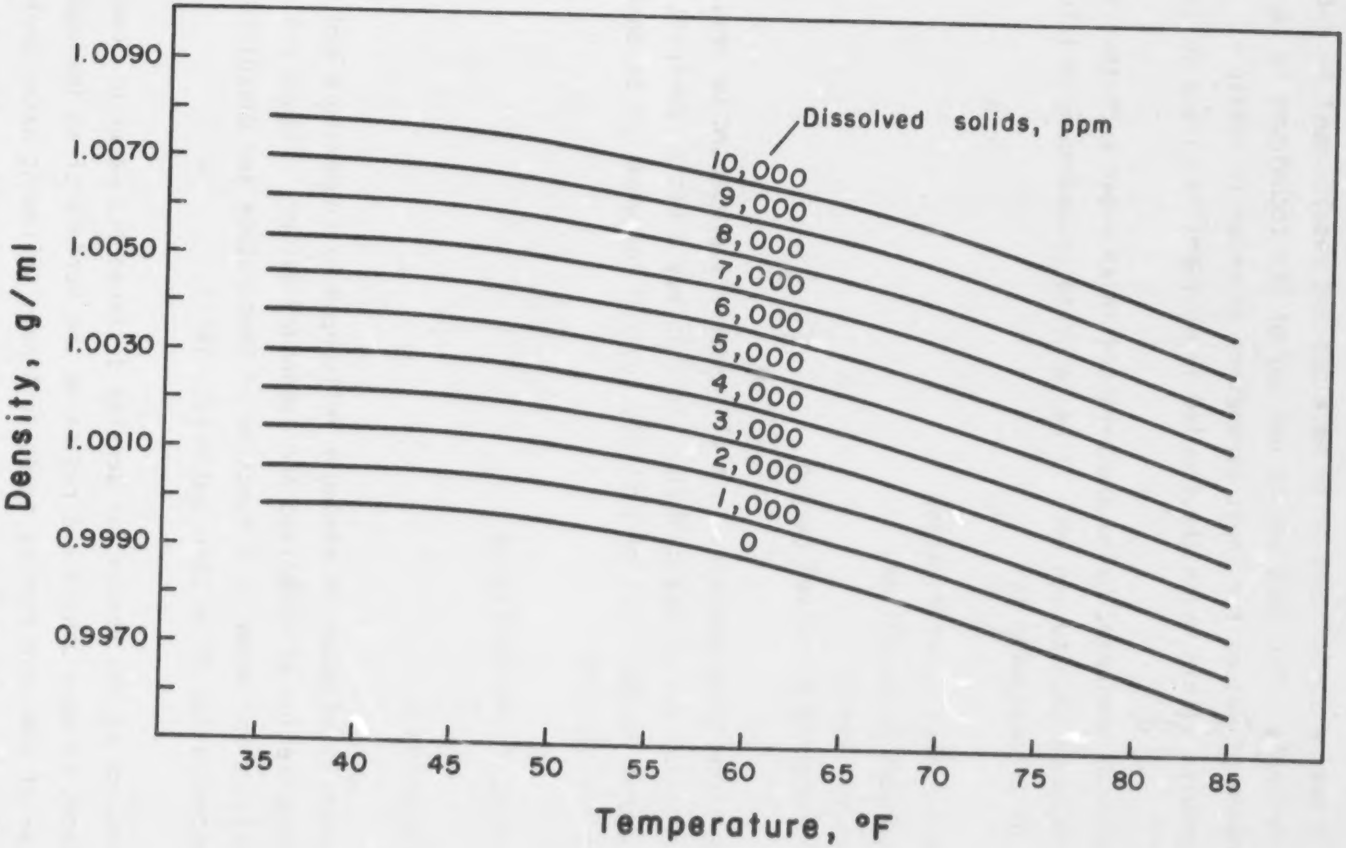
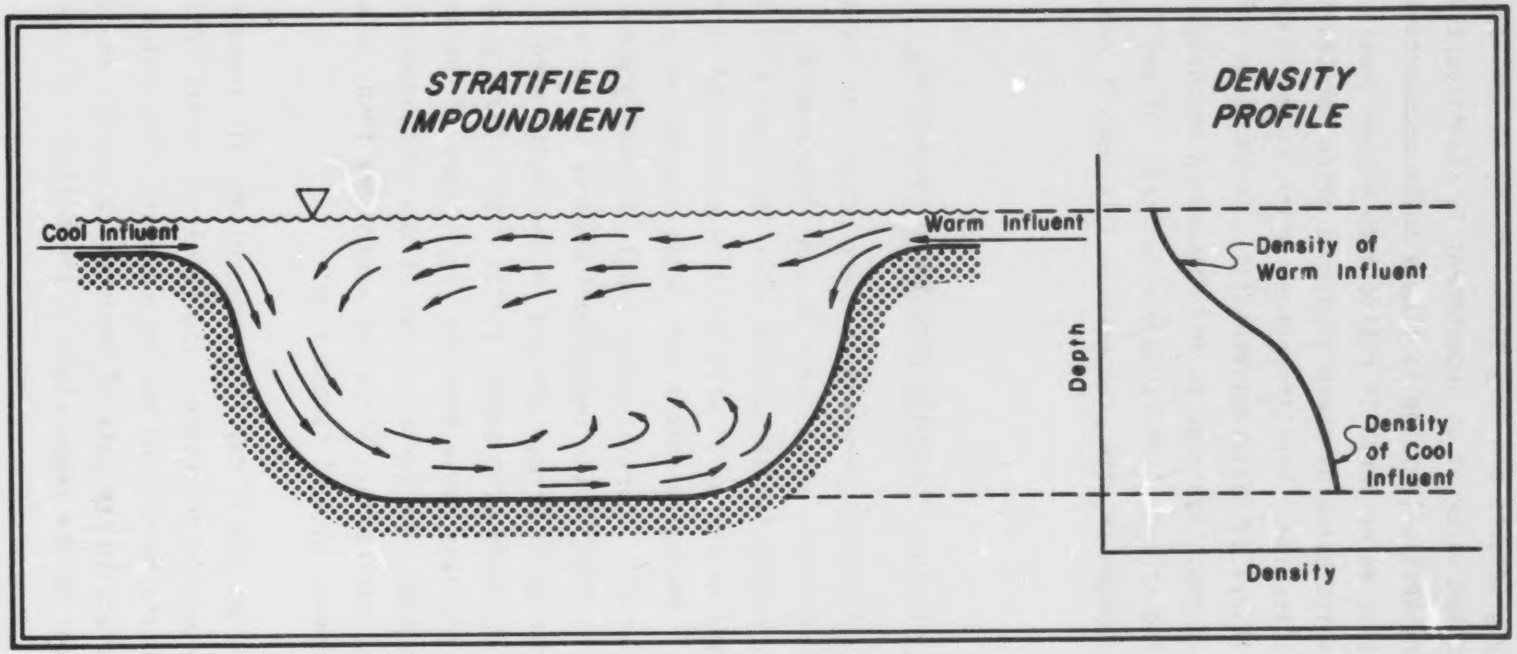


FIGURE V-1 WATER DENSITY AS A FUNCTION OF TEMPERATURE AND DISSOLVED SOLIDS CONCENTRATION (FROM CHEN AND ORLOB, 1973)



STRATIFIED IMPOUNDMENT
DENSITY PROFILE
Cool Influent
Warm Influent
Depth
Density of Warm Influent
Density of Cool Influent
Density

FIGURE V-2 WATER FLOWING INTO AN IMPOUNDMENT TENDS TO MIGRATE TOWARD A REGION OF SIMILAR DENSITY

Where density gradients are very steep, mixing is inhibited. Thus, where the bottom water of an impoundment is significantly more dense than surface water, vertical mixing is likely to be unimportant. The fact that low density water tends to reside atop higher density water and that mixing is inhibited by steep gradients often results in impoundment stratification. Stratification, which is the establishment of distinct layers of different densities, tends to be enhanced by quiescent conditions. Conversely, any phenomenon encouraging mixing, such as wind stress, turbulence due to large inflows or destabilizing changes in water temperature will tend to reduce or eliminate strata.

5.2.1.1 Annual Cycle in a Thermally Stratified Impoundment

Figure V-3 shows schematically the processes of thermal stratification and overturn which occur in many impoundments. Beginning at "a" in the figure (winter), cold water (at about 4°C) flows into the impoundment which may at this point be considered as fully mixed. There is no thermal gradient over depth and the impoundment temperature is about 6°C. During spring ("b"), inflowing water is slightly warmer than that of the impoundment because of the exposure of the tributary stream to warmer air and increasingly intense sunlight. This trend continues during the summer ("c"), with tributary water being much warmer and less dense than the deep waters of the impoundment. At the same time, the surface water of the impoundment is directly heated by insolation. Since the warm water tends to stay on top of the impoundment, thermal strata form.

As fall approaches ("d"), day length decreases, air temperatures drop, and solar intensity decreases. The result is cooler inflows and a cooling trend in the surface of the impoundment. The bottom waters lag behind the surface in the rate of temperature change, and ultimately the surface may cool to the temperature of the bottom. Since continued

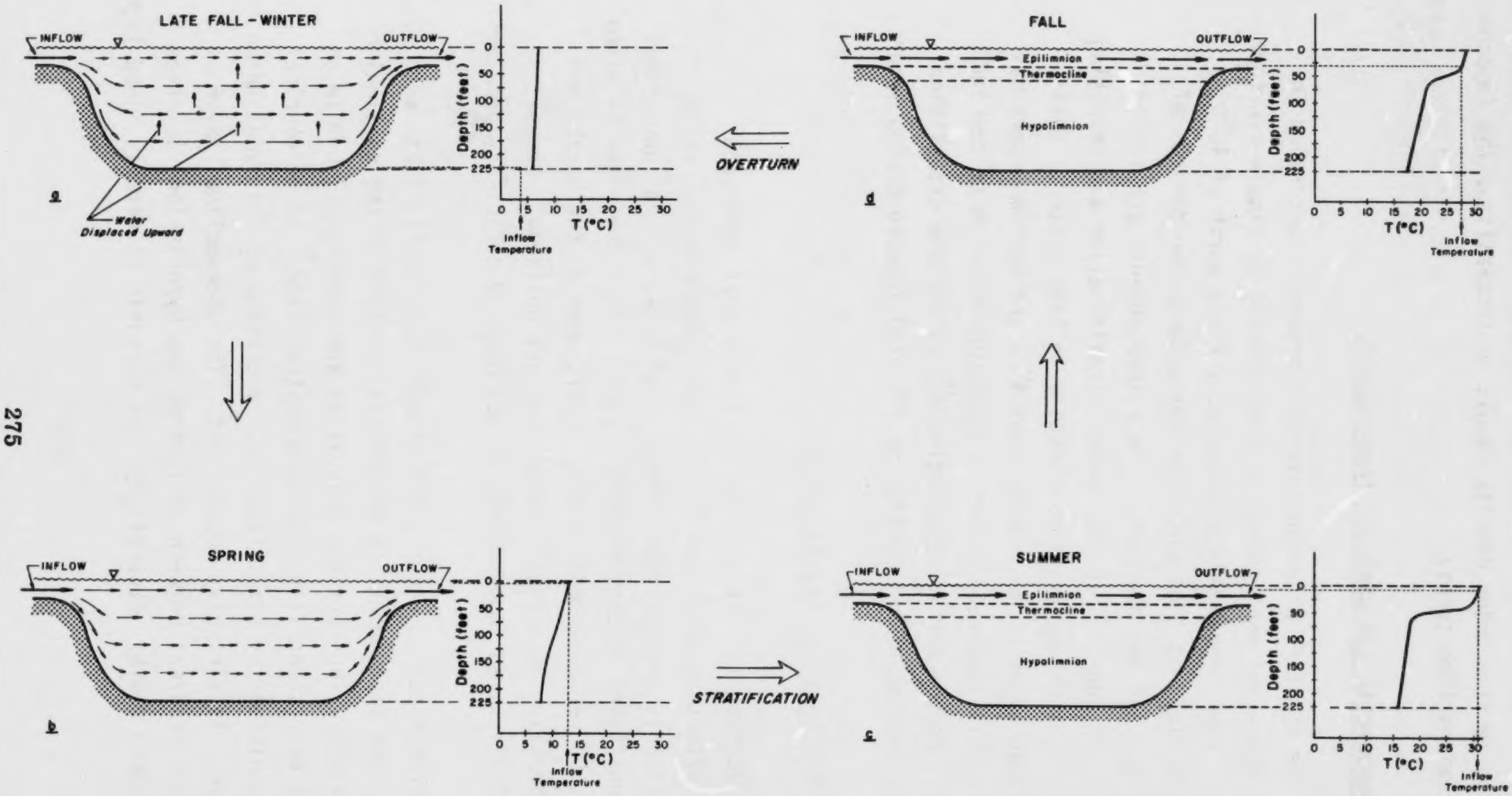

LATE FALL - WINTER
INFLOW
OUTFLOW
a
Water Displaced Upward
Depth (feet)
0
50
100
150
200
225
0 5 10 15 20 25 30
T (°C)
Inflow Temperature
OVERTURN
FALL
INFLOW
OUTFLOW
Epilimnion
Thermocline
Hypolimnion
d
Depth (feet)
0
50
100
150
200
225
0 5 10 15 20 25 30
T (°C)
Inflow Temperature
SPRING
INFLOW
OUTFLOW
b
Depth (feet)
0
50
100
150
200
225
0 5 10 15 20 25 30
T (°C)
Inflow Temperature
STRATIFICATION
SUMMER
INFLOW
OUTFLOW
Epilimnion
Thermocline
Hypolimnion
c
Depth (feet)
0
50
100
150
200
225
0 5 10 15 20 25 30
T (°C)
Inflow Temperature

FIGURE V-3 ANNUAL CYCLE OF THERMAL STRATIFICATION AND OVERTURN IN AN IMPOUNDMENT

increases in surface water density result in instability, the impoundment water mixes (overturns).

5.2.1.2 Monomictic and Dimictic Impoundments

The stratification and overturn processes described in Figure V-3 represent what occurs in a monomictic or single-overturn water body. Some impoundments, especially those north of 40°N latitude and those at high elevation may undergo two periods of stratification and two overturns. Such impoundments are termed "dimictic." In addition to the summer stratification and resulting fall overturn, such impoundments stratify in late winter. This occurs because water is most dense near 4°C, and bottom waters may be close to this temperature, while inflowing water is colder and less dense. As the surface goes below 4°C strata are established. With spring warming of the surface to 4°C wind induced mixing occurs.

5.2.1.3 Importance of Stratification

Stratification is likely to be the single most important phenomenon affecting water quality in many impoundments. Where stratification is absent, water mixes vertically, and net horizontal flow is significant to considerable depths. Since the water is mixed vertically, DO replenishment usually occurs even to the bottom and anoxic (literally "no oxygen") conditions are unlikely. Generally speaking, fully mixed impoundments do not have DO deficiency problems.

When stratification occurs, the situation is vastly different. Flow within the impoundment is essentially limited to the epilimnion (surface layer). Thus surface velocities are somewhat higher in an impoundment when stratified than when unstratified. Since vertical mixing is inhibited by stratification, reaeration of the hypolimnion (bottom layer) is virtually nonexistent. The thermocline (layer of steep thermal gradient between epilimnion and hypolimnion) is often at considerable depth. Accordingly, the euphotic (literally "good light") zone is likely to be limited to the epilimnion. Thus photosynthetic activity doesn't serve to reoxygenate the hypolimnion. The water that becomes the hypolimnion has some oxygen demand prior to the establishment of strata. Because bottom (bethic) matter exerts a further demand, and because some settling of particulate matter into the hypolimnion may occur, the D.O. level in the hypolimnion will gradually decrease over the period of stratification.

Anoxic conditions in the hypolimnion result in serious chemical and biological changes. Microbial activity leads to hydrogen sulfide (H_2S) evolution as well as formation of other highly toxic substances, and these may be harmful to indiginous biota.

It should be noted that the winter and spring strata and overturn are relatively unimportant here since the major concern is anoxic conditions in the hypolimnion in summer. Thus all impoundments will be considered as monomictic.

Strong stratification is also important in prediction of sedimentation rates and trap efficiency estimates. These topics are to be covered later.

5.2.2 Prediction of Thermal Stratification

Computation of impoundment heat influx is relatively straightforward, but prediction of thermal gradients is complicated by prevailing physical conditions, physical mixing phenomena, and impoundment geometry. Such factors as depth and shape of impoundment bottom, magnitude and configuration of inflows, and degree of shielding from the wind are very much more difficult to quantify than insolation, back radiation, and still air evaporation rates. Since the parameters which are difficult to quantify are critical to predicting stratification characteristics, no attempt has been made to develop a simple calculation procedure. Instead, a tested model (Chen and Orlob, 1975; Lorenzen and Fast, 1976) has been subjected to a sensitivity analysis and the results plotted to show thermal profiles over depth and over time for some representative geometries and climatological conditions. The plots are presented in Appendix D.

To assess thermal stratification in an impoundment, it is necessary only to determine which of the sets of plots most closely approximates conditions in the impoundment studied. Parameters which were varied to generate the plots and values used are shown in Table V-1.

Table V-2 shows the climatological conditions used to represent the geographic locales listed in Table V-1. For details of simulation technique, see Appendix E.

5.2.2.1 Using the Thermal Plots

Application of the plots to assess stratification characteristics begins with determining reasonable values for the various parameters listed in Table V-1. For geographic locale, the user should determine whether the impoundment of interest is near one of the ten areas for which thermal plots have been generated. If so, then the set of plots for that area should be used. If the impoundment is not near one of the ten areas, then the user may obtain data for the parameters listed in Table V-2 (climatologic data) and then select the modeled locale which best matches the region of interest.

Next, the user must obtain geometric data for the impoundment. Again, if the impoundment of interest is like one for which plots have been generated, then that set should be used. If not, the user should bracket the studied impoundment. As an example, if the studied impoundment is 55 feet deep (maximum), with a surface area of about 4×10^7 feet^2, then the 40 and 75 foot deep impoundment plots should be used.

Mean hydraulic residence time (τ_w) may be estimated using the mean total inflow rate (Q) and the impoundment volume (V):

TABLE V-1

PARAMETER VALUES USED IN GENERATION OF THERMAL GRADIENT PLOTS (APPENDIX D)

Parameter			Value
Geographic Locale			Atlanta, Georgia Billings, Montana Burlington, Vermont Flagstaff, Arizona Fresno, California Minneapolis, Minnesota Salt Lake City, Utah San Antonio, Texas Washington, D.C. Wichita, Kansas
Geometry	Depth (maximum, feet)	Surface Area (feet^2)	Volume (feet^3)
	20	8.28×10^6	7.66×10^7
	40	3.31×10^7	6.13×10^8
	75	1.16×10^8	4.04×10^9
	100	2.07×10^8	9.58×10^9
	200	8.28×10^8	7.66×10^{10}
Mean Hydraulic Residence Time			Days
			10 30 75 100 200 ∞
Wind Mixing*			High Low

*See Appendix E.

TABLE V-2

TEMPERATURE, CLOUD COVER, AND DEW POINT DATA FOR THE TEN GEOGRAPHIC LOCALES USED TO DEVEOP THERMAL STRATIFICATION PLOTS (APPENDIX D). SEE FOOT OF TABLE FOR NOTES.

	Temperature (°F)			Dew	Cloud Cover	Wind
	Max.	Mean	Min.	Point (°F)	Fraction	(MPH)
		Atlanta (Lat:33.8°N, Long:84.4°W)				
January	54	45	36	34	.63	11
February	57	47	37	34	.62	12
March	63	52	41	39	.61	12
April	72	61	50	48	.55	11
May	81	70	57	57	.55	9
June	87	77	66	65	.58	8
July	88	79	69	68	.63	8
August	88	78	68	67	.57	8
September	83	73	63	62	.53	8
October	74	63	52	51	.45	9
November	62	51	40	40	.51	10
December	53	44	35	34	.62	10
		*Billings (Lat:45.8°N, Long:108.5°W)				
January	27	18	9	11	.68	13
February	32	22	12	16	.68	12
March	38	27	16	20	.71	12
April	51	38	26	28	.70	12
May	60	47	34	38	.64	11
June	68	54	40	46	.60	11
July	79	63	46	48	.40	10
August	78	61	45	46	.42	10
September	67	52	37	38	.54	10
October	55	42	30	31	.56	11
November	38	29	20	22	.66	13
December	32	22	14	15	.66	13

TABLE V-2 - CONT.

	Temperature (°F)			Dew	Cloud Cover	Wind
	Max.	Mean	Min.	Point (°F)	Fraction	(MPH)
Burlington (Lat:44.5°N, Lat:73.2°W)						
January	27	18	9	12	.72	10
February	29	19	10	12	.69	10
March	38	29	20	20	.66	10
April	53	43	33	32	.67	10
May	67	56	44	43	.67	9
June	54	66	77	54	.61	9
July	82	71	59	59	.58	8
August	80	68	57	58	.57	8
September	71	60	49	51	.60	8
October	59	49	39	40	.65	9
November	44	38	29	30	.79	10
December	31	23	15	17	.78	10
Flagstaff (Lat:35.2°N, Long:111.3°W)						
January	40	27	14	14	.59	8
February	43	30	17	16	.49	9
March	50	36	22	17	.50	11
April	59	43	28	20	.49	12
May	68	51	34	22	.41	11
June	77	60	42	25	.24	11
July	81	66	50	43	.54	9
August	79	64	49	43	.53	9
September	75	59	42	35	.29	8
October	63	47	31	25	.31	8
November	51	36	21	20	.34	8
December	44	30	17	15	.44	7

TABLE V-2 CONT.

	Temperature (°F)			Dew Point (°F)	Cloud Cover Fraction	Wind (MPH)
	Max.	Mean	Min.			
	Fresno (Lat:36.7°N, Long:119.8°W)					
January	55	46	37	38	.67	6
February	61	51	40	41	.61	6
March	68	55	42	41	.53	7
April	76	61	46	44	.44	7
May	85	68	52	45	.34	8
June	92	75	57	48	.19	8
July	100	81	63	51	.11	7
August	98	79	61	52	.11	6
September	92	74	56	51	.15	6
October	81	65	49	46	.28	5
November	68	54	40	42	.44	5
December	57	47	38	40	.70	5
	Minneapolis (Lat:45.0°N, Long:93.3°W)					
January	22	12	3	6	.65	11
February	26	16	5	10	.62	11
March	37	28	18	20	.67	12
April	56	45	33	32	.65	13
May	70	58	46	43	.64	12
June	79	67	56	55	.60	11
July	85	76	61	60	.49	9
August	82	71	59	59	.51	9
September	72	61	49	50	.51	10
October	60	48	37	40	.54	11
November	40	31	21	25	.69	12
December	27	18	9	13	.69	11

TABLE V-2 CONT.

	Temperature (°F)			Dew Point (°F)	Cloud Cover Fraction	Wind (MPH)
	Max.	Mean	Min.			
Salt Lake City (Lat:40.8°N, Long:111.9°W)						
January	37	27	18	20	.69	7
February	42	33	23	23	.70	8
March	51	40	30	26	.65	9
April	62	50	37	31	.61	9
May	72	58	45	36	.54	10
June	82	67	52	40	.42	9
July	92	76	61	44	.35	9
August	90	75	59	45	.34	10
September	80	65	50	38	.34	9
October	66	53	39	34	.43	9
November	49	38	28	28	.56	8
December	40	23	32	24	.69	7
San Antonio (Lat:29.4°N, Long:98.5°W)						
January	62	52	42	39	.64	9
February	66	55	45	42	.65	10
March	72	61	50	45	.63	10
April	79	68	58	55	.64	11
May	85	75	65	64	.62	10
June	92	82	72	68	.54	10
July	94	84	74	68	.50	10
August	94	84	73	67	.46	8
September	89	79	69	65	.49	8
October	82	71	60	56	.46	8
November	70	59	49	46	.54	9
December	65	42	54	41	.57	9

TABLE V-2 CONT.

	Temperature ($^{\circ}$F)			Dew	Cloud Cover	Wind
	Max.	Mean	Min.	Point ($^{\circ}$F)	Fraction	(MPH)
	Washington, D.C. (Lat:38.9°N, Long:77.0°W)					
January	44	37	30	25	.61	11
February	46	38	29	25	.56	11
March	54	45	36	29	.56	12
April	60	56	46	40	.54	11
May	76	66	56	52	.54	10
June	83	74	65	61	.51	10
July	87	78	69	65	.51	9
August	85	77	68	64	.51	8
September	79	70	61	59	.48	9
October	68	59	50	48	.47	9
November	57	48	39	36	.54	10
December	46	43	31	26	.58	10
	Wichita (Lat:37.7°N, Long:97.3°W)					
January	42	32	22	21	.50	12
February	47	36	26	25	.51	13
March	56	45	33	30	.52	15
April	68	57	45	41	.53	15
May	77	66	55	53	.53	13
June	88	77	65	62	.46	13
July	92	81	69	65	.39	12
August	93	81	69	53	.38	11
September	84	71	59	55	.39	12
October	72	60	48	43	.40	12
November	34	55	44	33	.44	13
December	45	36	27	25	.50	12

TABLE V-2 CONT.

Notes: Mean: Normal daily average temperature, °F.
Max.: Normal daily maximum temperature, °F.
Min.: Normal daily minimum temperature, °F.
Wind: Mean wind speed, MPH
Dew Point: Mean dew point temperature, °F.

*Complete data were not available for Billings. Tabulated data are actually a synthesis of available data for Billings, Montana and Yellowstone, Wyoming.

All data taken from Climatic Atlas of the U.S., 1974.

$$\tau_w = V/Q \qquad (V\text{-}1)$$

Again, the sets of plots bracketing the value of τ_w should be examined. Where residence times are greater than 200 days, the residence time has little influence on stratification (as may be verified in Appendix D) and either the 200 day or infinite time plots mav be used.

Finally, the wind mixing coefficient was used to generate plots for windy areas (high wind) and for very well protected areas (low wind). The user must judge where his studied impoundment falls and interpolate in the plots accordingly. (See Appendix E).

EXAMPLE V-1

Thermal Stratification

Suppose one wants to know the likelihood that hypothetical Limpid Lake is stratified during June. The first step is to compile the physical conditions for the lake in terms of the variables listed in Table V-1. Table V-3 shows how this might be done. Next, refer to the indexes provided in Appendix D to locate the plot set for conditions most similar to those of the studied impoundment. In this case, the Wichita plots for a 200-foot deep impoundment with no inflow and high mixing rate would be chosen (see Table V-3). Figure V-4 is a reproduction of the appropriate page from Appendix D.

TABLE V-3

LIMPID LAKE CHARACTERISTICS

Item	Limpid Lake	Available Plot
Location	Manhattan, Kansas	Wichita, Kansas
Depth, ft (maximum)	180	200
Volume, ft^3	6×10^{10}	7.66×10^{10}
Mean residence time (τ_w)	500 days	∞ (no inflow)
Mixing	high (windy)	high coefficient

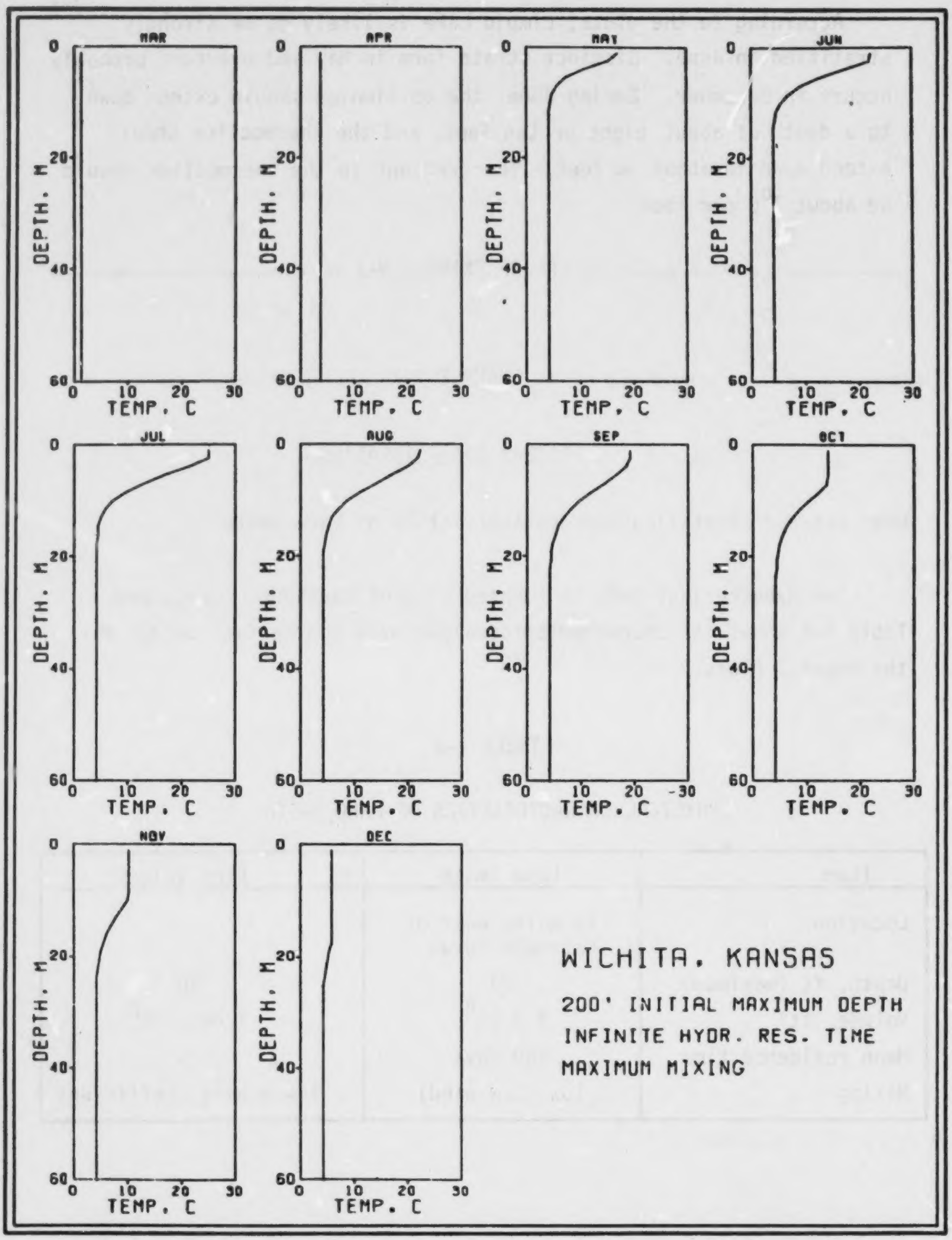


FIGURE V-4 THERMAL PROFILE PLOTS USED IN EXAMPLE V-1

According to the plots, Limpid Lake is likely to be strongly stratified in June. Distinct strata form in May and overturn probably occurs in December. During June, the epilimnion should extend down to a depth of about eight or ten feet, and the thermocline should extend down to about 30 feet. The gradient in the thermocline should be about 1°C per foot.

END OF EXAMPLE V-1

EXAMPLE V-2

Thermal Stratification

What are the stratification characteristics of Lake Smith?

The hypothetical lake is located east of Carthage, Texas, and Table V-4 shows its characteristics along with appropriate values for the thermal plots.

TABLE V-4

PHYSICAL CHARACTERISTICS OF LAKE SMITH

Item	Lake Smith	Plot Values
Location	15 miles east of Carthage Texas	-
Depth, ft (maximum)	23	20
Volume, ft^3	3×10^8	1.66×10^8
Mean residence time	250 days	∞
Mixing	low (low wind)	low mixing coefficient

From the available data for Lake Smith, it appears that plots for a 20-foot deep impoundment with no inflow and low mixing coefficient should give a good indication of the degree of summertime stratification. The one remaining problem is climate. Data for nearby Shreveport, Louisiana compare well with those of Atlanta (Table V-5), for which plots are provided in Appendix D, and latitudes are similar. Shreveport is somewhat warmer and insolation is higher, but this is a relatively uniform difference over the year. The net effect should be to shift the thermal plots to a slightly higher temperature but to influence the shape of the plots and the timing of stratification little. As a result, the plots for Atlanta may be used, bearing in mind that the temperatures are likely to be biased uniformly low. Figure V-5 (reproduced from Appendix D) shows thermal plots for a 20-foot deep Atlanta area impoundment having no significant inflow and low wind stress. From the figure, it is clear that the lake is likely to stratify from April or May through September, the epilimnion will be very shallow, and the thermocline will extend down to a depth of about 7 feet. The thermal gradient is in the range of about 7^oC per meter, as an upper limit, during June. Bottom water warms slowly during the summer until the impoundment becomes fully mixed in October.

END OF EXAMPLE V-2

5.3 SEDIMENT ACCUMULATION

5.3.1 Introduction

Reservoirs, lakes, and other impoundments are usually more quiescent than tributary streams, and thus act as large settling basins for suspended particulate matter. Sediment deposition in impoundments gradually diminishes water storage capacity to the point where lakes fill in and reservoirs become useless. In some cases, sediment accumulation may reduce the useful life of a reservoir to as little as ten to twenty years (Marsh, et al., 1975).

TABLE V-5

COMPARISON OF MONTHLY CLIMATOLOGIC DATA
FOR SHREVEPORT, LOUISIANA AND ATLANTA, GEORGIA
DATA ARE PRESENTED AS SHREVEPORT/ATLANTA
(CLIMATIC ATLAS OF THE U.S., 1974)

	Temperature, °F			Dew Point, °F	Cloud Cover, Fraction	Wind, MPH
	Max.	Mean	Min.			
January	57/54	48/45	38/36	38/34	.60/.63	9/11
February	60/57	50/47	41/37	40/34	.58/.62	9/12
March	67/63	57/52	47/41	44/39	.54/.61	10/12
April	75/72	65/61	55/50	54/48	.50/.55	9/11
May	83/81	73/70	63/57	62/57	.48/.55	9/9
June	91/87	81/77	71/66	69/65	.44/.58	8/8
July	92/88	82/79	72/69	71/68	.46/.63	7/8
August	94/88	83/78	73/68	70/67	.40/.57	7/8
September	88/83	78/73	67/63	65/62	.40/.53	7/8
October	79/74	67/63	55/52	55/51	.38/.45	7/9
November	66/62	55/51	45/40	45/40	.46/.51	8/10
December	59/53	50/44	40/35	39/34	.58/.62	9/10

Shreveport Lat:32.5°N, Long:94°W

Atlanta Lat:33.8°N, Long:84.4°W,

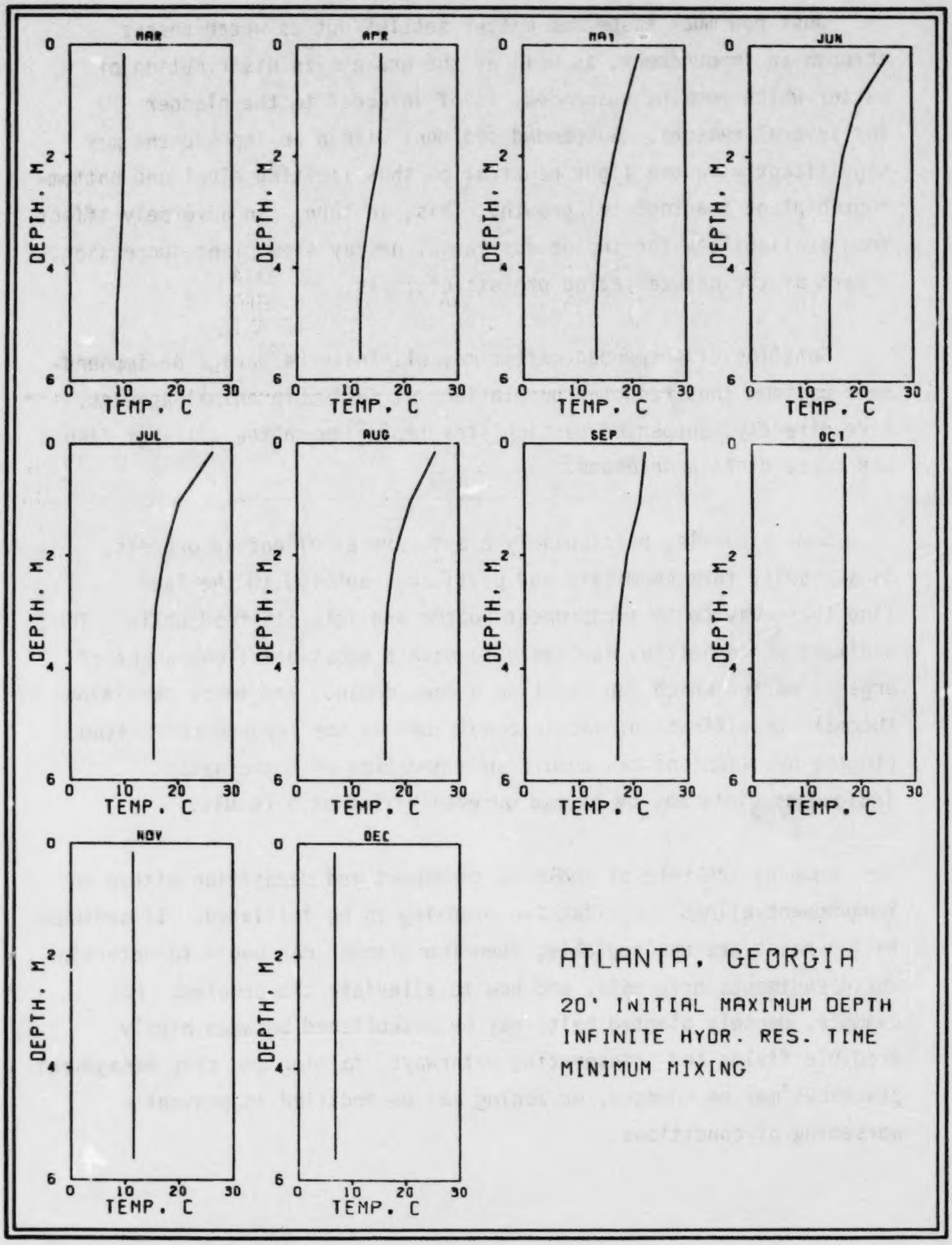


FIGURE V-5 THERMAL PROFILE PLOTS APPROPRIATE FOR USE IN EXAMPLE V-2

Just how much suspended matter settles out as water passes through an impoundment, as well as the grain size distribution of matter which remains suspended, is of interest to the planner for several reasons. Suspended sediment within an impoundment may significantly reduce light penetration thus limiting algal and bottom-rooted plant (macrophyte) growth. This, in turn, can adversely affect food availability for indigenous fauna, or may slow plant succession, a part of the natural aging process of lakes.

Settling of suspended matter may eliminate harborage on impoundment bottoms thus reducing populations of desirable animal species. More directly, suspended particulates impinging on the gills of fish may cause disease or death.

Some minerals, particularly clays, are excellent adsorbents. As a result, farm chemicals and pesticides applied to the land find their way to an impoundment bottom and into its food chain. The sediment which settles is likely to have a substantial component of organic matter which can exert an oxygen demand, and under conditions of thermal stratification, anoxic conditions on the impoundment bottom (in the hypolimnion) can result in generation of toxic gases. Indigenous biota may be harmed or even killed as a result.

Knowing the rate of sediment transport and deposition within an impoundment allows for effective planning to be initiated. If sedimentation rates are unacceptable, then the planner can begin to determine where sediments originate, and how to alleviate the problem. For example, densely planted belts may be established between highly erodible fields and transporting waterways, farming and crop management practices may be changed, or zoning may be modified to prevent a worsening of conditions.

These considerations, along with others relating to sediment carriage and deposition in downstream waterways, make estimates of sedimentation rates of interest here. Impoundment sediment computation methods discussed in this section will permit the planner to estimate annual impoundment sediment accumulation as well as short term accumulation (assuming constant hydraulic conditions). Application of the methods will permit the planner to estimate the amount of sediment removed from transport in a river system due to water passage through any number of impoundments.

5.3.2 Annual Sediment Accumulation

5.3.2.1 Use of Available Data

Data provided in Appendix F permit estimation of annual sediment accumulation in acre-feet for a large number of impoundments in the U.S. The data and other materials presented provide some basic impoundment statistics useful to the planner in addition to annual sediment accumulation rates.

To use Appendix F, first determine which impoundments within the study area are of interest in terms of annual sediment accumulation. Refer to the U.S. map included in the appendix and find the index numbers of the region within which the impoundment is located. The data tabulation in the appendix is arranged in order of index number, and this will permit quick location of specific data. Referring to the impoundment data section of the appendix, total annual sediment accumulation in acre feet is given by multiplying average annual sediment accumulation in acre feet per square mile of net drainage area ("Annual Sediment Accum.") by the net drainage area ("Area") in square miles:

$$\text{Total Accumulation} = \text{Annual Sediment Accum.} \times \text{Area} \qquad (V\text{-}2)$$

To convert to average annual loss of capacity expressed as a percent, divide total annual accumulation by storage capacity (from Appendix F), and multiply by 100. Note that this approach, as well as those presented later, do not account for packing of the sediment under its own weight. This results in an overestimate in loss of capacity. Note also that other data in Appendix F may be of interest in terms of drainage area estimates for determining river sediment loading and assessment of storm water sediment transport on an annual basis.

5.3.2.2 Trap Efficiency and the Ratio of Capacity to Inflow

Where data are not available in Appendix F for a specific impoundment, the following method will permit estimation of annual or short-term sediment accumulation rates. The method is only useful, however, for normal ponded reservoirs.

To use this approach, a suspended sediment rating curve should be obtained for tributaries to the impoundment. An example of a sediment rating curve is provided in Figure V-6.

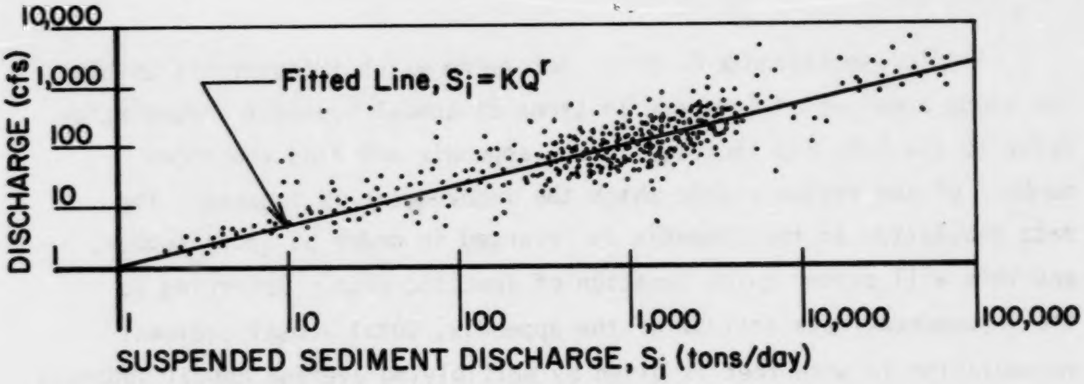


FIGURE V-6 SEDIMENT RATING CURVE SHOWING SUSPENDED SEDIMENT DISCHARGE AS A FUNCTION OF FLOW (AFTER LINSLEY, KOHLER, AND PAULHUS, 1958)

On the basis of such a curve, one can estimate the mean sediment mass transport rate (S_i) in mass per unit time for tributaries. If neither rating curve nor data are available, one may estimate sediment transport rates on a basis of data from nearby channels, compensating for differences by using mean velocities. To a first approximation, it would be expected that:

$$S_i \approx S_j\left(\frac{V_i}{V_j}\right)^2 \qquad \text{(V-3)}$$

where

S_i = sediment transport rate to be determined in tributary "i" in mass per unit time,

S_j = known transport rate for comparable tributary (j) in same units as S_i,

V_i = mean velocity for tributary i over the time period, and

V_j = mean velocity in tributary j over the same time period as V_i

Once average transport rates over the time period of interest have been determined, the proportion, and accordingly the weight of sediment settling out in the impoundment may be estimated. Figure V-7 is a graph showing the relationship between percent of sediment trapped in an impoundment versus the ratio of capacity to inflow rate. The implicit relationship is:

$$P = f(C/Q_i) \qquad \text{(V-4)}$$

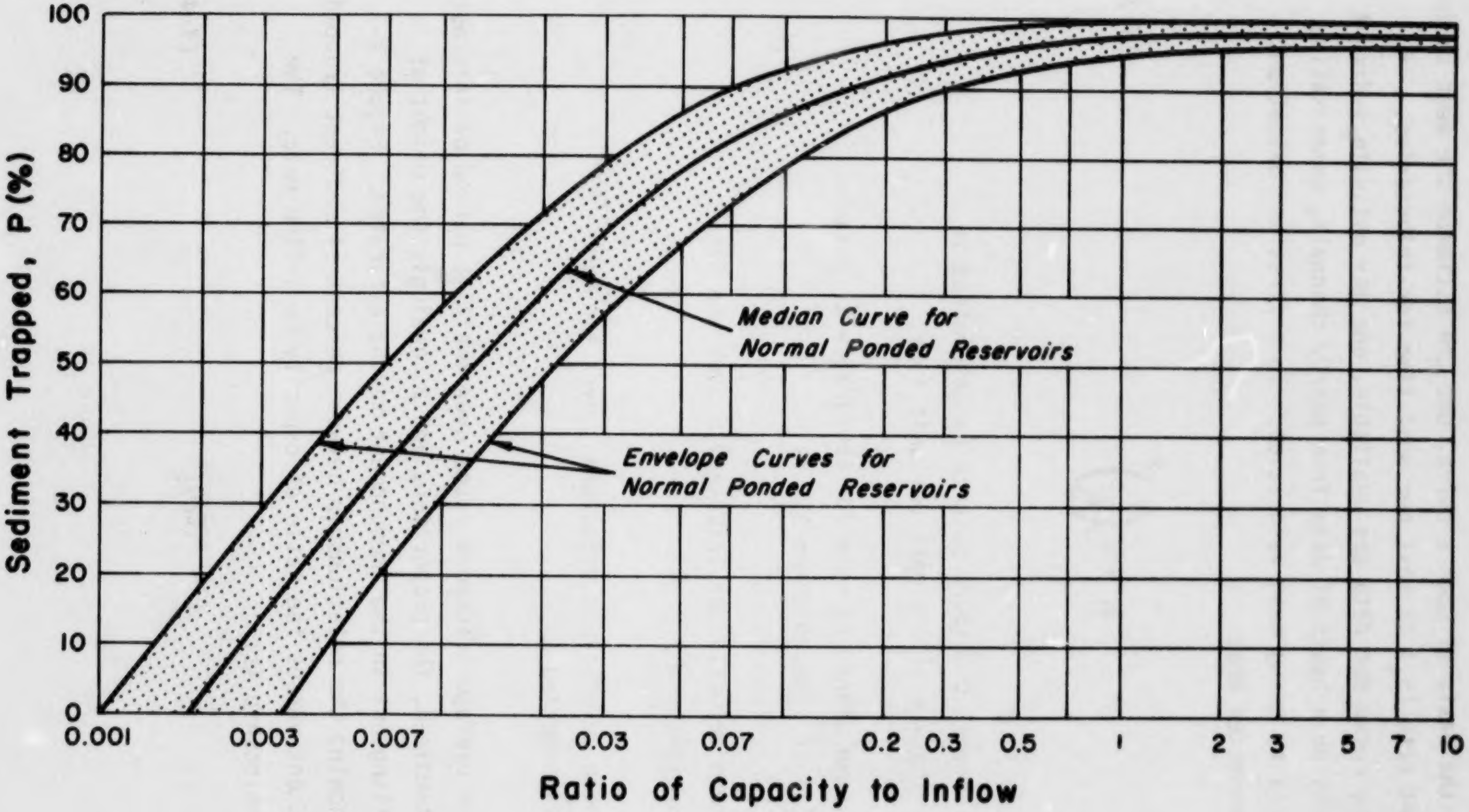


FIGURE V-7 RELATIONSHIP BETWEEN THE PERCENTAGE OF INFLOW-TRANSPORTED SEDIMENT RETAINED WITHIN AN IMPOUNDMENT AND RATIO OF CAPACITY TO INFLOW (LINSLEY, KOHLER, AND PAULHUS, 1958)

where

P = percent of inflowing sediment trapped

C = capacity of the impoundment in acre-feet, and

Q_i = water inflow rate in acre-feet per year

Data used for development of the curves in Figure V-7 included 41 impoundments of various sizes throughout the U.S. (Linsley, Kohler, and Paulhus, 1958).

To estimate the amount of suspended sediment trapped within an impoundment using this method, the capacity of the impoundment in acre-feet must first be determined. Next, average annual inflow, or better, average flow for the time period of interest is estimated.

Then,

$$S_t = S_i P \tag{V-5}$$

where

S_t = weight of sediment trapped per time period t

P = trap efficiency (expressed as a decimal) from Figure V-7

A word of caution is in order here. The above described techniques for evaluating sediment deposition in impoundments are capable of providing reasonable estimates, but only where substantial periods of time are involved - perhaps 6 months or longer. The methods may be used for shorter study periods, but results must then be taken only as very rough estimates, perhaps order-of-magnitude.

5.3.3 Short-Term Sedimentation Rates

The three-step procedure presented below provides a means to make short-term sediment accumulation rate estimates for storm-event analysis and to estimate amounts of different grain-size fractions passing through an impoundment. The steps are:

- Determine terminal fall velocities for the grain size distribution
- Estimate hydraulic residence time
- Compute trap (sedimentation) rate

5.3.3.1 Fall Velocity Computation

When a particle is released in standing water, it will remain roughly stationary if its density equals that of the water. If the two densities differ, however, the particle will begin to rise or fall relative to the water. It will then tend to accelerate until the drag force imposed by the water exactly counterbalances the force accelerating the particle. Beyond this point, velocity is essentially constant, and the particle has reached terminal velocity. For spheres of specific gravity greater than 1, Stokes' law expresses the relationship between fall velocity (terminal velocity) and several other physical parameters of water and the particle.

$$v_{max} = \frac{g}{18\mu}(\rho_p - \rho_w)d^2 = \frac{1}{18\mu}(D_p - D_w)d^2 \qquad \text{(V-6)}$$

where

v_{max} = terminal velocity of the spherical particle (ft sec^{-1})
g = acceleration due to gravity (32.2 ft sec^{-2})
ρ_p = mass density of the particle (slugs ft^{-3})
ρ_w = mass density of water (slugs ft^{-3})
d = particle diameter (ft)
μ = absolute viscosity of the water (lb $sec \cdot ft^{-2}$)
D_p = weight density of particle (lb ft^{-3})

D_w = weight density of water (lb ft^{-3})

Stokes' law is satisfactory for Reynolds numbers between 1×10^{-4} and 0.5 (Camp, 1963). Reynolds number is given by:

$$R = \frac{vd}{\nu} \qquad (V\text{-}7)$$

where

R = Reynolds number

v = particle velocity

ν = kinematic viscosity of water

Generally, for particles of diameter less than 3×10^{-2} inches (0.7 mm) this criterion is met. For large particles, how far conditions deviate from this may be observed using the following approach (Camp, 1968). According to Newton's law for drag, drag force on a particle is given by:

$$F_d = CA\rho_w v^2/2 \qquad (V\text{-}8)$$

where

F_d = the drag force

C = unitless drag coefficient

A = projected area of the particle in the direction of motion

Equating the drag force to the gravitational (driving) force for the special case of a spherical particle, velocity is given by:

$$v_{max} = \sqrt{\frac{4g\,(\rho_p - \rho_w)d}{3C \quad \rho_w}} \qquad (V\text{-}9)$$

All variables in the expression for v_{max} (Equation V-9) may be easily estimated except C, since C is dependent upon Reynold's number. According to Equation (V-7), Reynolds number is a function of v. Thus a "trial and error" or iterative procedure would ordinarily be necessary to estimate C. However, a somewhat simpler approach is available to evaluate the drag coefficient and Reynolds number. First, estimate CR^2 using the expression (Camp, 1968):

$$CR^2 = 4\rho_w\,(\rho_p - \rho_w)\,gd^3/3\mu^2 \qquad (V\text{-}10)$$

Then, using the plot in Figure V-8, estimate R and then C. For R>0.1 use of Equation (V-9) will give better estimates of V_{max} than will Equation (V-6).

Generally, one of the two approaches for spherical particles will give good estimates of particle fall velocity in an effectively laminar flow field (in impoundments). Occasionally, however, it may prove desirable to compensate for nonsphericity of particles. Figure V-9, which shows the effect of particle shape on the drag coefficient C, may be used to do this. Note that for R<1, shape of particle does not materially affect C, and no correction is necessary.

A second problem in application of the Newton/Stokes approach described above is that it does not account for what is called hindrance. Hindrance occurs when the region of fluid surrounding a falling particle is disrupted (by the particle motion) and the velocity of other nearby particles is thereby decreased. Figure V-10 shows this effect schematically.

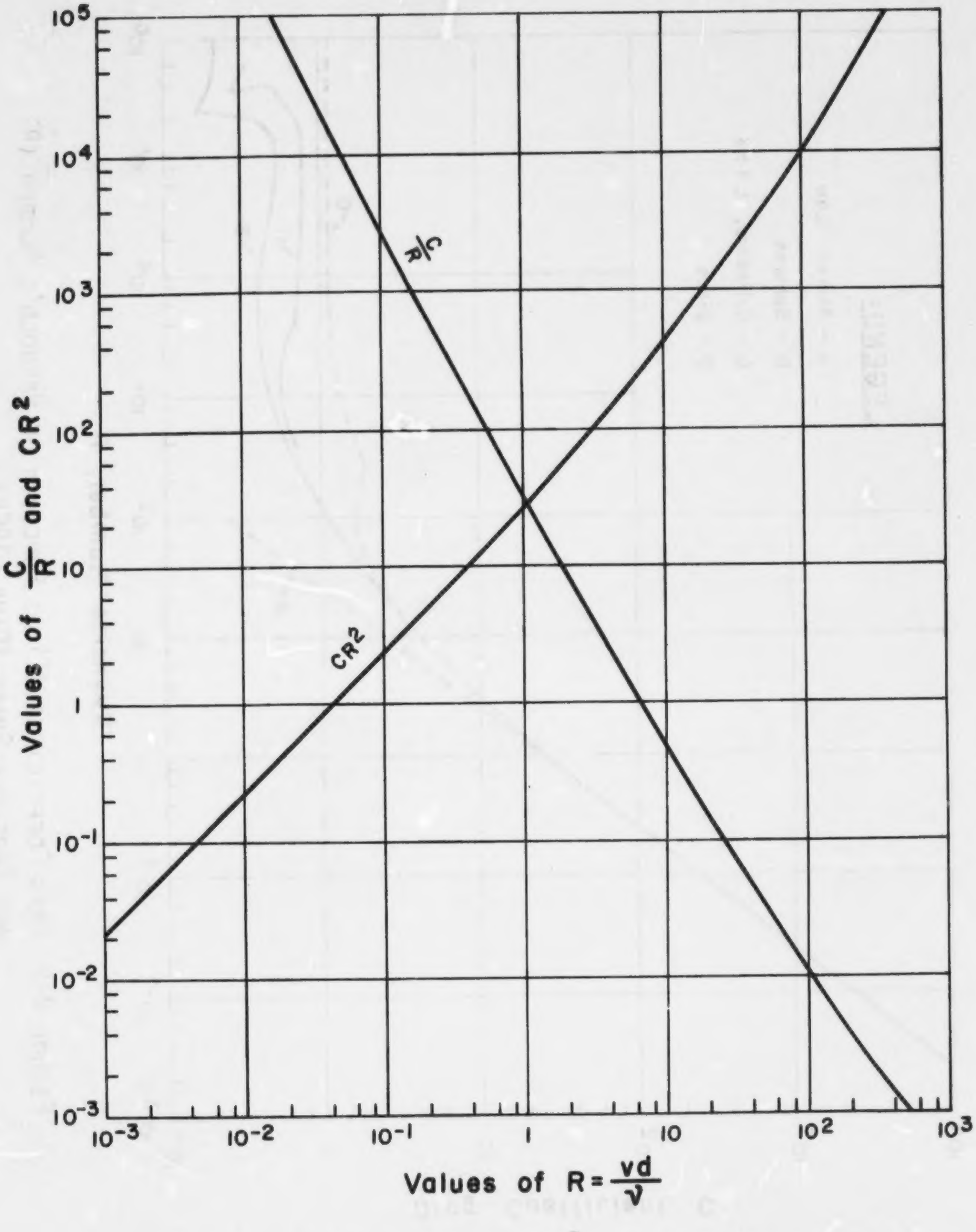


FIGURE V-8 PLOT OF C/R AND CR^2 VERSUS R (CAMP, 1968)

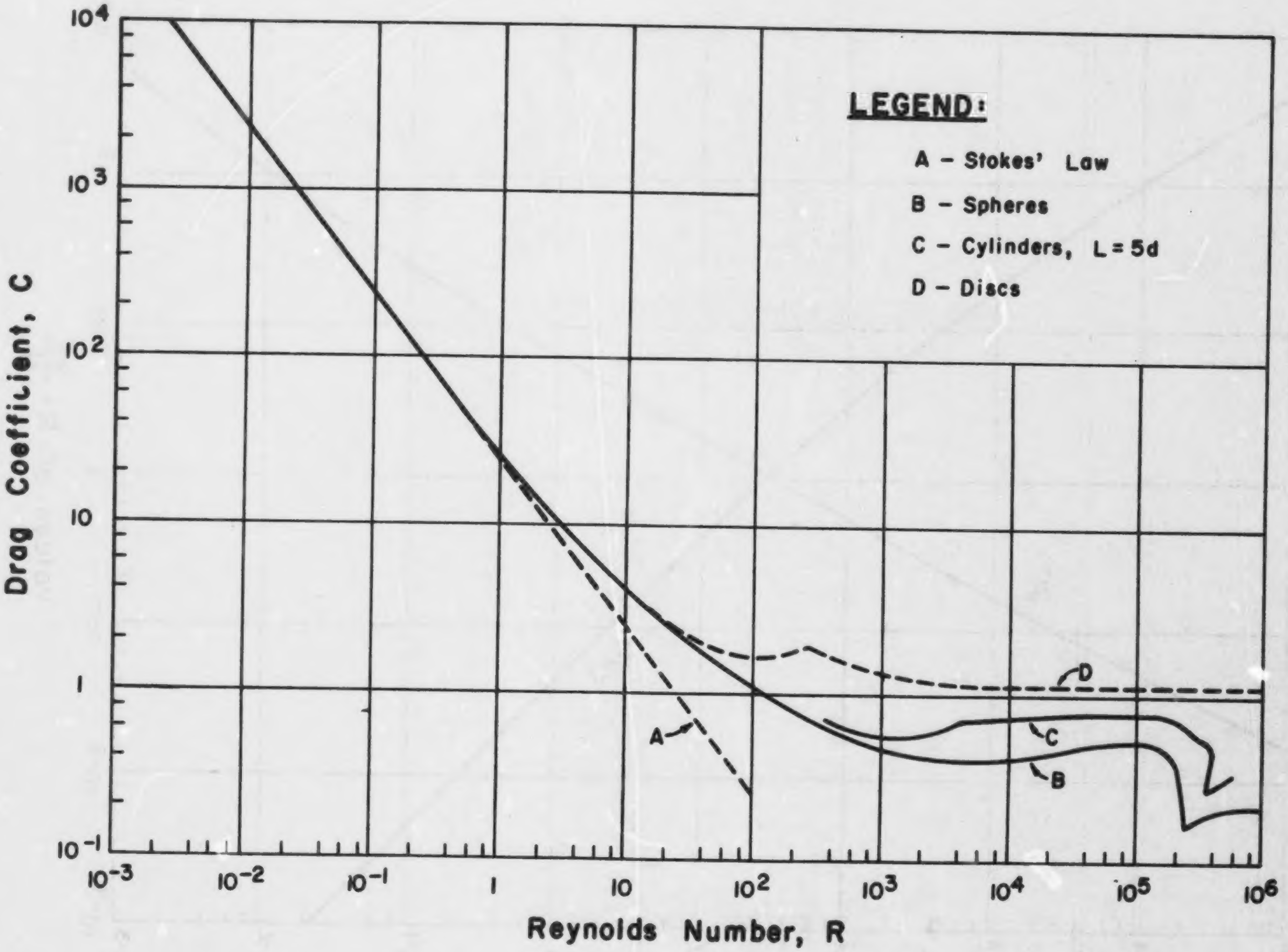


FIGURE V-9 DRAG COEFFICIENT (C) AS FUNCTION OF REYNOLD'S NUMBER (R) AND PARTICLE SHAPE (CAMP, 1968)

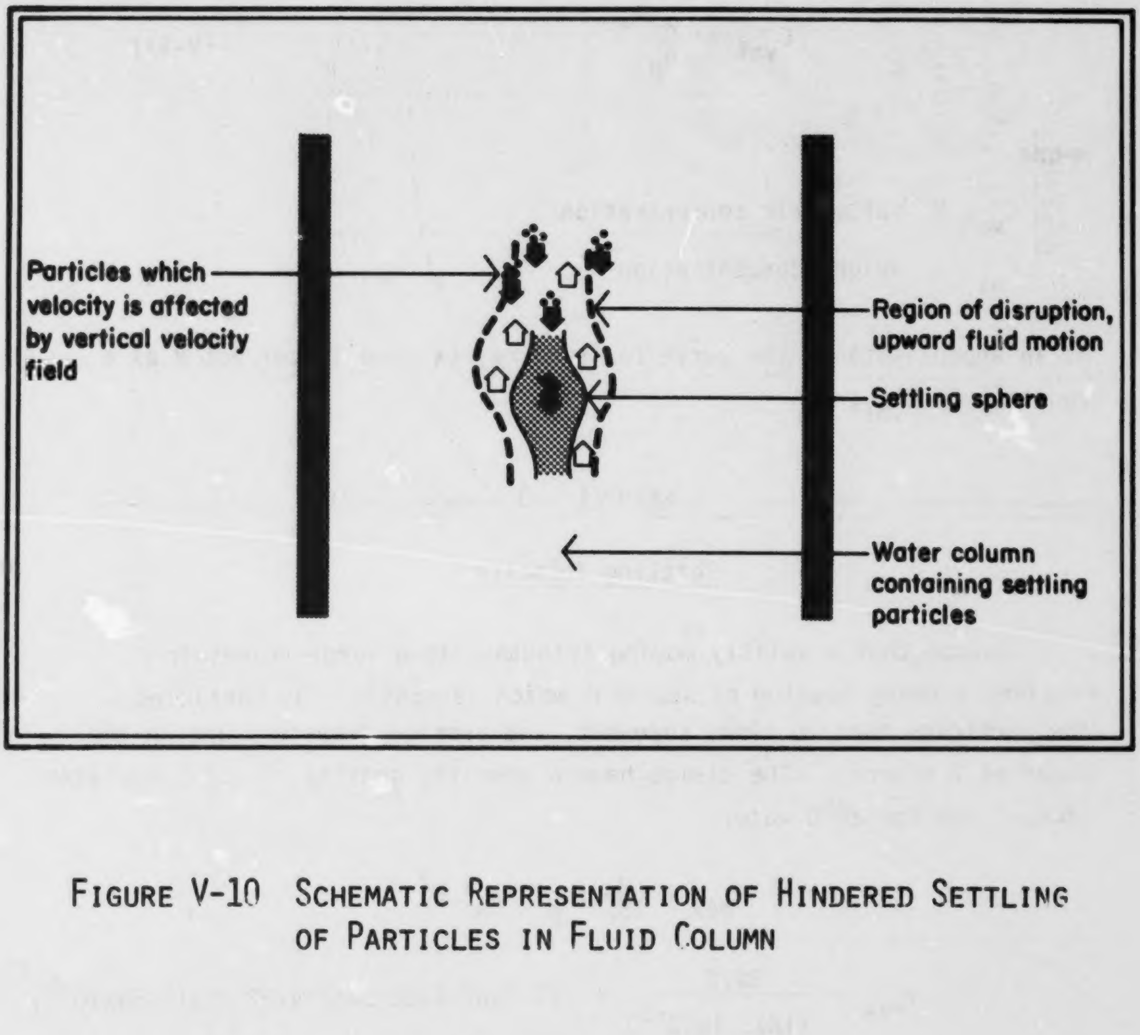

Particles which velocity is affected by vertical velocity field
Region of disruption, upward fluid motion
Settling sphere
Water column containing settling particles

FIGURE V-10 SCHEMATIC REPRESENTATION OF HINDERED SETTLING OF PARTICLES IN FLUID COLUMN

A very limited amount of research has been done to determine the effect of particle concentration on fall velocity.(Camp, 1968). Some data have been collected however, and Figure V-11 is a plot of a velocity correction factor, v'/v as a function of volumetric concentration. Volumetric concentration is given by:

$$C_{vol} = \frac{C_{wt}\rho_w}{\rho_p} \qquad \text{(V-11)}$$

where

C_{vol} = volumetric concentration

C_{wt} = weight concentration

As an approximation, the curve for sand may be used to correct v as a function of C_{vol}.

EXAMPLE V-3

Settling Velocity

Assume that a swiftly moving tributary to a large reservoir receives a heavy loading of sediment which is mostly clay particles. The particles tend to clump somewhat, and average diameters are on the order of 2 microns. The clumps have a specific gravity of 2.2. Applying Stokes' law for 20°C water,

$$v_{max} = \frac{g}{18\mu}(\rho_p - \rho_w)\, d^2$$

$$v_{max} = \frac{32.2}{(18\text{x}2.1\text{x}10^{-5})} \times (2.2\text{x}62.4/32.2-62.4/32.2)\text{x}(6.56\text{x}10^{-6})^2$$

$$= 8.53\text{x}10^{-6} \text{ ft sec}^{-1} = .03 \text{ ft hr}^{-1}$$

Thus the particles of clay might be expected to fall about 9 inches per day in the reservoir. It should be noted that for such a low

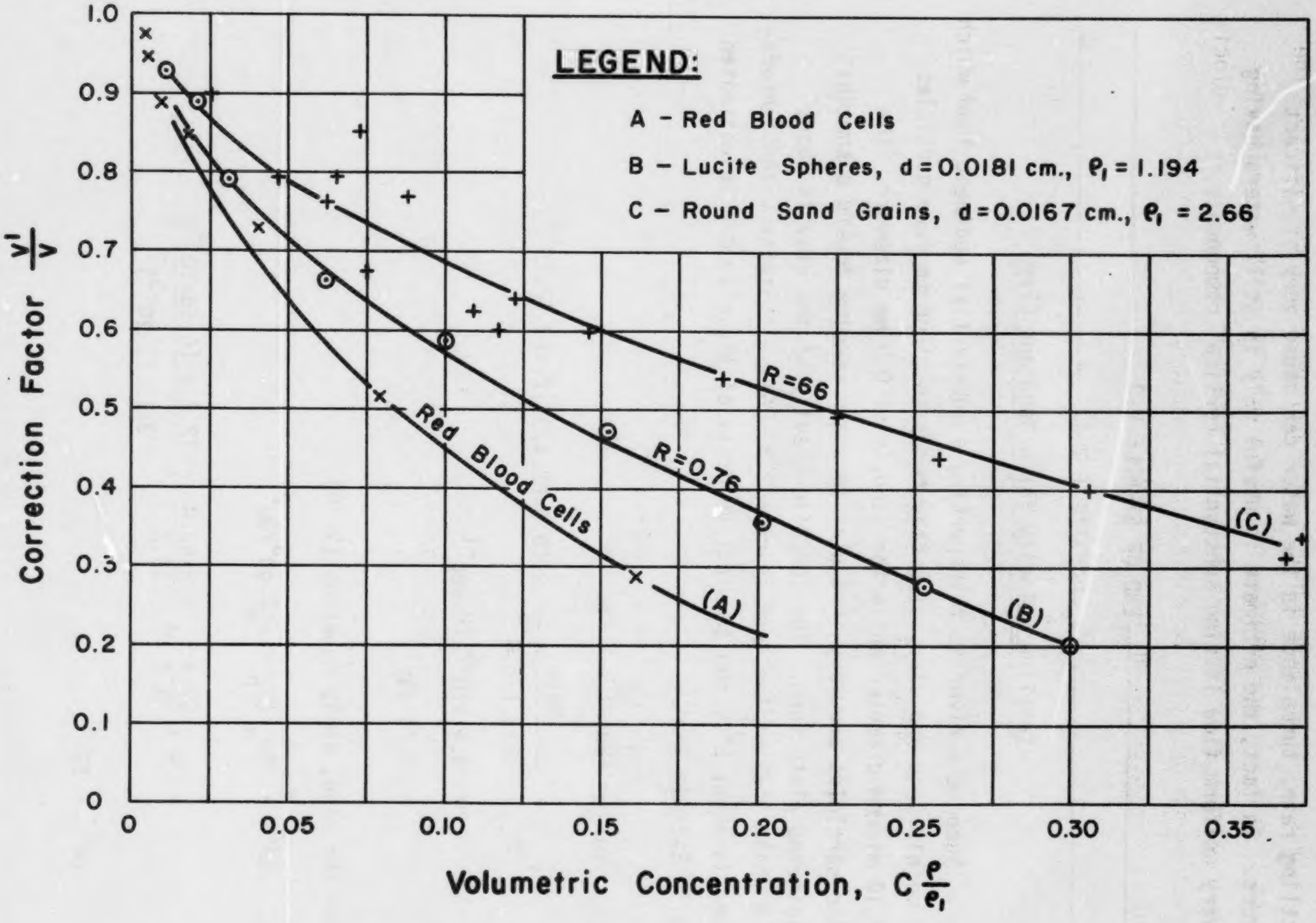


FIGURE V-11 VELOCITY CORRECTION FACTOR FOR HINDERED SETTLING (FROM CAMP, 1968)

settling rate, turbulence in the water can cause very significant errors. In fact, the estimate is useful only in still waters having a very uniform flow lacking substantial vertical components.

END OF EXAMPLE V-3

EXAMPLE V-4

Settling Velocity for a Sand and Clay

Suppose a river is transporting a substantial sediment load which is mainly sand and clay. The clay tends to clump to form particles of 10 micron diameter while the sand is of 0.2 mm diameter. The sand particles are very irregular in shape tending toward a somewhat flattened plate form. The specific gravity of the clay is about 1.8 while that of the sand is near 2.8. Given that the water temperature is about $5^{o}C$, the terminal velocity of the clay may be estimated as in Example V-3:

$$v_{max} = \frac{g}{18\mu} (\rho_p - \rho_w) d^2$$

$$v_{max} = \frac{32.2}{18 x 3.17 x 10^{-5}} \times (0.8 x 62.4/32.2) x (3.28 x 10^{-5})^2$$

$$= 9.4 \times 10^{-5} \text{ ft sec}^{-1}$$

$$\simeq 8 \text{ ft day}^{-1}$$

For the sand, apply Equation (V-10)

$$CR^2 = 4\rho_w (\rho_p - \rho_w) gd^3/3\mu^2$$

$$= 4 \times \frac{62.4}{32.2} \times \frac{1.8 x 62.4}{32.2} \times \frac{32.2 \times (6.56 x 10^{-4})^3}{3 x (3.17 x 10^{-5})^2}$$

$$CR^2 = 82$$

Referring to Figure V-8, a value of CR^2 equal to 82 represents R≈2.8 and C≈10.3. From Figure V-9, the corrected drag coefficient for discs is close to 10.3 (no correction really necessary). Then, using Equation (V-9) as an approximation,

$$v_{max} = \sqrt{\frac{49\ (\rho_p - \rho_w)\ d}{3C\rho_w}}$$

$$v_{max} = \sqrt{\frac{4x32.2x(1.8x62.4/32.2)x6.56x10^{-4}}{3x10.3x62.4/32.2}}\ .$$

$$v_{max} = 0.07\ \text{ft sec}^{-1} = 252\ \text{ft hr}^{-1}$$

Thus the clay will settle about 8 feet per day while the sand will settle about 6048 feet per day (252 feet per hour).

——————————— END OF EXAMPLE V-4 ———————————

5.3.4 Impoundment Hydraulic Residence Time

Once settling velocities have been estimated for selected grain sizes, the final preparatory step in estimating sediment deposition rates is to compute hydraulic residence time.

Hydraulic residence time represents the mean time a particle of water resides within an impoundment. It is not, as is sometimes thought, the time required to displace all water in the impoundment with new. In some impoundments, inflowing water may be conceptualized as moving in a vertical plane from inflow to discharge. This is called plug flow. In long, narrow, shallow impoundments with high inflow velocities, this is often a good assumption. As discussed later, however, adoption of this model leads to another problem, namely, is water within the plug uniform or does sediment concentration vary over depth within the plug?

A second model assumes that water flowing into an impoundment instantaneously mixes laterally with the entire receiving layer. The layer may or may not represent the entire impoundment depth. This simplification is often a good one in large surfaced, exposed impoundments having many small inflows.

Regardless of the model assumed for the process by which water traverses an impoundment from inflow to discharge, hydraulic residence time is computed as in Equation (V-1). That is,

$$\tau_w = V/Q$$

The only important qualification is that to be meaningful, V must be computed taking account of stagnant areas, whether these are regions of the impoundment isolated from the main flow by a sand spit or promontory, or whether they are isolated by a density gradient, as in the thermocline and hypolimnion. Ignoring stagnant areas may result in a very substantial overestimate of τ_w, and in sediment trap computations, an overestimate in trap efficiency. Actually τ_w computed in this way is an adjusted hydraulic residence time. All references to hydraulic residence time in the remainder of this chapter mean adjusted τ_w.

Hydraulic residence time is directly influenced by such physical variables as impoundment depth, shape, side slope, and shoaling, as well as hydraulic characteristics such as degree of mixing, stratification, and flow velocity distributions. The concepts involved in evaluating many of these factors are elementary. The evaluation itself is complicated, however, by irregularities in impoundment shape and data inadequacies. Commonly, an impoundment cannot be represented well by a simple 3-dimensional figure, and shoaling and other factors may restrict flow to a laterally narrow swath through the water body.

In most cases, hydraulic residence time may be estimated, although it is clear that certain circumstances tend to make the computation error-prone. The first step in the estimation process is to obtain impoundment inflow, discharge, and thermal regime data as well as topographic/bathymetric maps of the system. Since a number of configuration types are possible, the methods are perhaps best explained using examples.

EXAMPLE V-5

Hydraulic Residence Time in Unstratified Impoundments

The first step in estimating hydraulic residence time for purposes of sedimentation analysis is to determine whether there are significant stagnant areas. These would include not only regions cut off from the main flow through the body, but also layers isolated by dense strata. Consequently, it must be determined whether or not the impoundment stratifies. Consider Upper Lake located on the Carmans River, Long Island, New York. The lake and surrounding region are shown in Figure V-12, and hypothetical geometry data are presented in Table V-6. Based upon Upper Lake's shallowness, its long, narrow geometry, and high tributary inflows, it is safe to assume that Upper Lake is normally unstratified. Also, because of turbulence likely at the high flows, one can assume that the small sac northeast of the discharge is not stagnant and that Upper Lake represents a slow moving river reach. With these assumptions, the computation of hydraulic residence time is as shown in Table V-6.

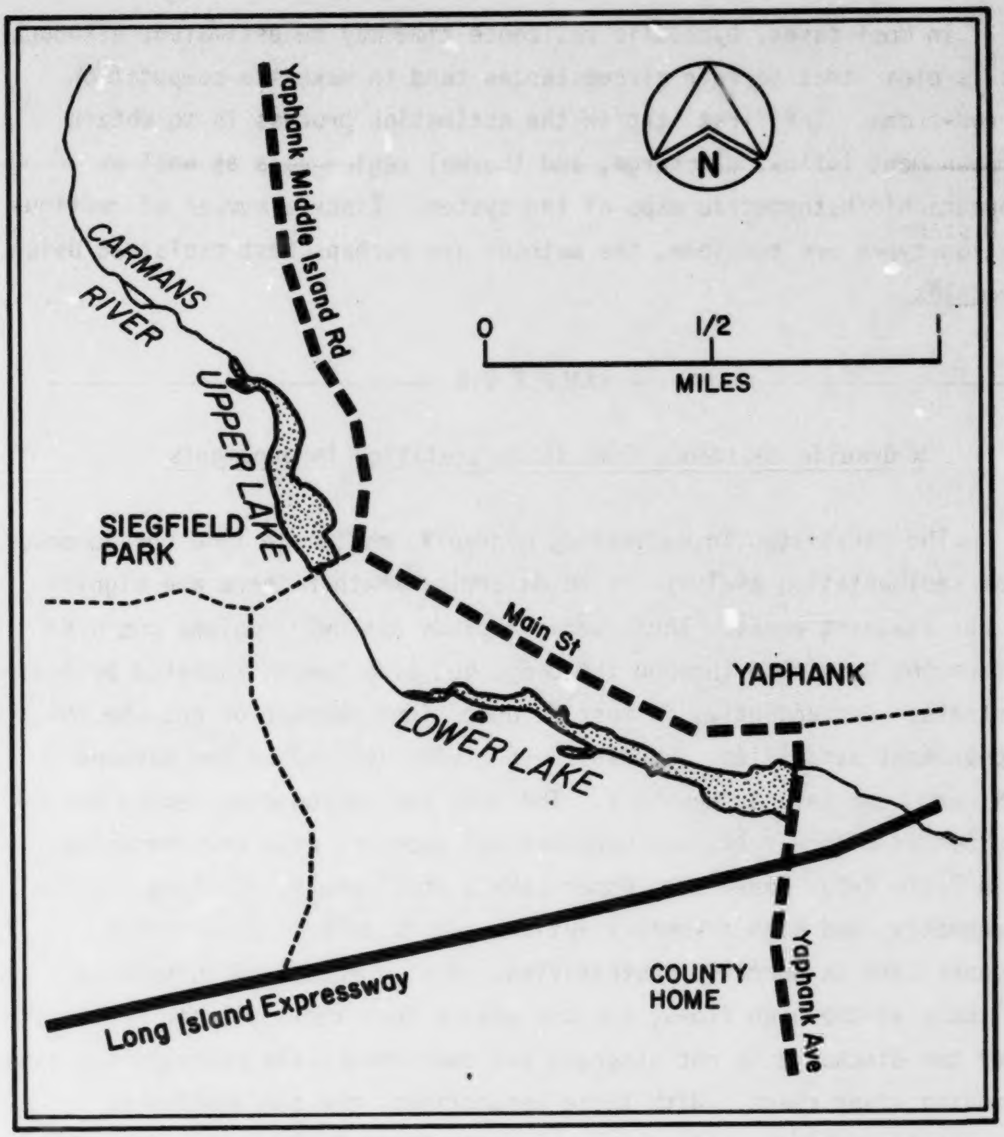

Yaphank Middle Island Rd
N
CARMANS RIVER
0
1/2
1
MILES
UPPER LAKE
SIEGFIELD PARK
Main St
YAPHANK
LOWER LAKE
COUNTY HOME
Yaphank Ave
Long Island Expressway

FIGURE V-12 UPPER AND LOWER LAKES AND ENVIRONS, LONG ISLAND, NEW YORK

TABLE V-6

HYPOTHETICAL PHYSICAL CHARACTERISTICS OF UPPER LAKE, BROOKHAVEN, SUFFOLK COUNTY, NEW YORK

Distance Downstream from Inflow Miles (feet)	D Average Depth ft.	W Average Width ft.	CSA Cross-sectional Area, D x W ft^2
0.05 (264)	3	63	189
0.10 (528)	4	110	440
0.15 (792)	6	236	1,416
0.20 (1,056)	7	315	2,205
0.25 (1,320)	7	340	2,380
0.30 (1,584)	8	315	2,520
0.35 (1,848)	7	550	3,850
0.40 (2,112)	8	550	4,400
0.45 (2,376)	7	354	2,478
0.50 (2,640)	10	350	3,500

Total length = 0.5 mi. (2,640 ft.) mean CSA = 2,338 ft^2

$$\left.\begin{array}{l}\text{Inflow from upstream} = 380\text{ cfs}\\ \text{Outflow to downstream} = 380\text{ cfs}\end{array}\right\}\text{ (steady-state)}$$

Computation

Volume (Vol) = Total length x mean cross-sectional area

$$\text{Vol} = 2{,}640\text{ ft.} \times 2{,}338\text{ ft}^2 = 6.17 \times 10^6\text{ ft}^3$$

Residence time (τ_w) = Vol/flow

$$\tau_w = 6.17 \times 10^6\text{ ft}^3/(380\text{ ft}^3/\text{sec}) = 1.62 \times 10^4\text{ sec (4.5 hr)}$$

Velocity (Vel) = length/τ_w

$$\text{Vel} = 2{,}640\text{ ft}/1.62 \times 10^4\text{ sec} = .163\text{ ft/sec}$$

Also shown in Figure V-12 is Lower Lake. According to the hypothetical data presented in Table V-7, Lower Lake is much deeper than Upper Lake. Its volume is significantly greater also, but the inflow rate is similar. In this case, particularly during the summer, it should be determined if the lake stratifies. For this example, however, we will assume that the time of the year makes stratification very unlikely, and that Lower Lake is a slow moving river reach. We then compute hydraulic residence time as shown in Table V-7. Figure V-13 in particular diagram 1, shows what these assumptions mean in terms of a flow pattern for both lakes.

END OF EXAMPLE V-5

EXAMPLE V-6

Assume for this example that Lower Lake is stratified during the period of interest. This significantly changes the computation of residence time. To a first approximation, one can merely revise the effective depth of the impoundment to be from the surface to the upper limit of the thermocline rather than to the bottom. Figure V-13 shows schematically what this simple model suggests for Lower Lake as a stratified impoundment (diagram 2 or possibly 3). The figure also shows wind-driven shallow, and deep impoundments. To the right of each diagram is a plot of the temperature profile over depth. Actually, the profile could represent a salinity gradient as well as a thermal gradient.

Table V-8 shows the procedure to estimate travel time for stratified Lower Lake. The upper boundary of the thermocline is assumed to be at a depth of 10 feet. For all later computations of sediment accumulation, this same 10 foot depth would be adopted. Such an assumption is valid presuming that the thermocline and hypolimnion are relatively quiescent. Thus once a particle enters the thermocline it can only settle, and can not leave the impoundment.

END OF EXAMPLE V-6

TABLE V-7

HYPOTHETICAL PHYSICAL CHARACTERISTICS OF LOWER LAKE, BROOKHAVEN, SUFFOLK COUNTY, NEW YORK

Distance Downstream from Inflow Miles (feet)	D Average Depth ft.	W Average Width ft.	CSA Cross-sectional Area, D x W ft^2
0.075 (396)	15	157	2,355
0.150 (792)	20	165	3,300
0.225 (1,188)	20	173	3,460
0.300 (1,584)	25	197	4,925
0.375 (1,980)	35	197	6,895
0.450 (2,376)	30	228	6,840
0.525 (2,772)	35	232	8,120
0.600 (3,168)	35	197	6,895
0.675 (3,564)	40	220	8,800
0.750 (3,960)	42	315	13,230
0.825 (4,356)	41	433	17,753
0.900 (4,752)	51	591	30,141
0.975 (5,148)	42	551	23,142
1.050 (5,544)	40	433	17,320
1.125 (5,940)	37	323	11,951

Total length = 1.125 mi (5,940 ft.) mean CSA = 11,008

Inflow from upstream 400 cfs
Outflow to downstream 390 cfs } (surface rising)

Average flow = 395 cfs

Computation

Volume (Vol) = Total length x mean cross-sectional area

$$\text{Vol} = 5{,}940 \text{ ft.} \times 11{,}008 \text{ ft}^2 = 6.54 \times 10^7 \text{ ft}^3$$

Residence Time (τ_w) = Vol/flow

$$\tau_w = 6.54 \times 10^7/(395 \text{ ft}^3/\text{sec}) = 1.65 \times 10^5 \text{ sec (46 hr)}$$

Velocity (Vel) = length/τ_w

$$\text{Vel} = 5{,}940 \text{ ft}/1.65 \times 10^5 \text{ sec} = .036 \text{ ft/sec}$$

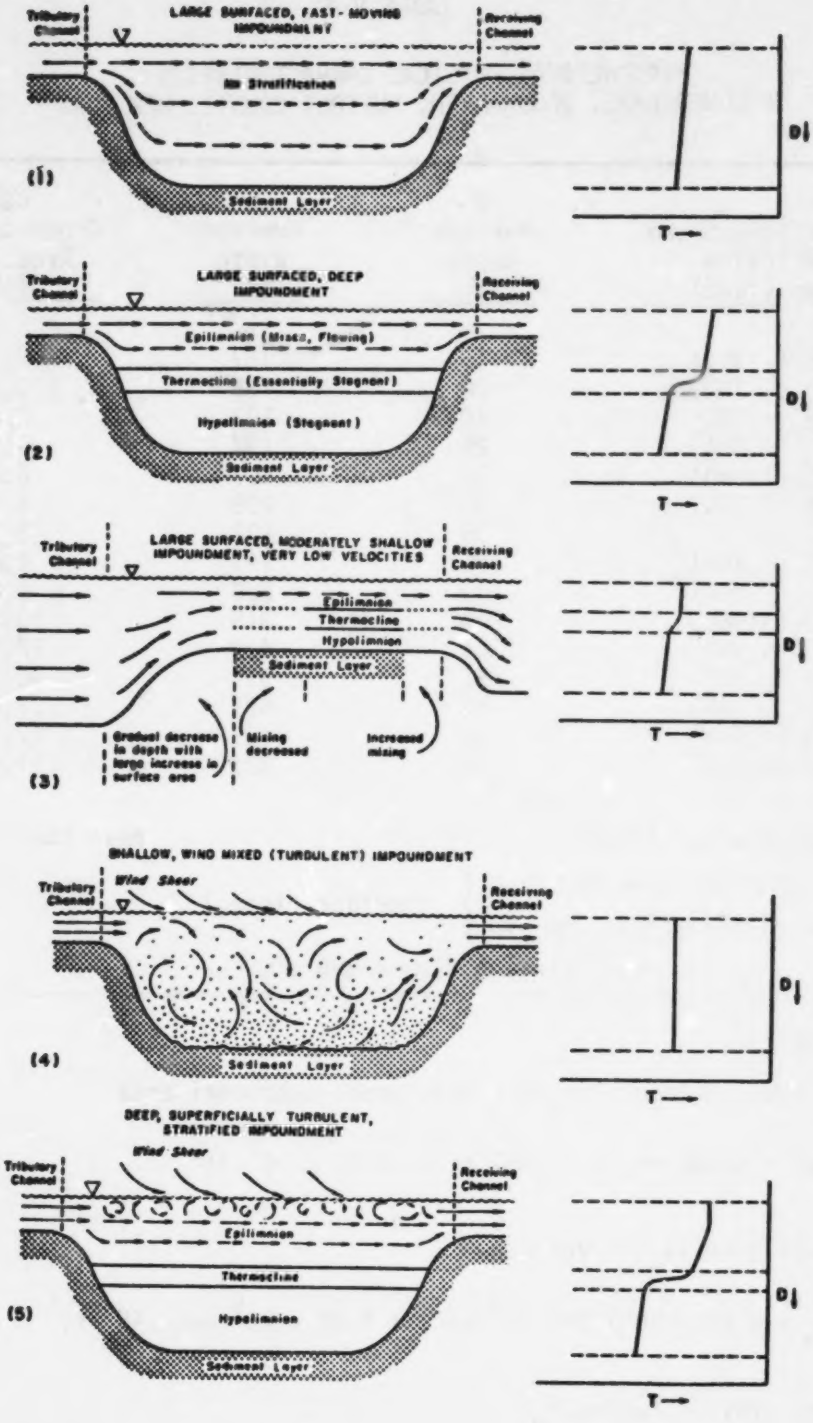

LARGE SURFACED, FAST-MOVING IMPOUNDMENT
Tributary Channel
Receiving Channel
No Stratification
Sediment Layer
(1)
D
T
LARGE SURFACED, DEEP IMPOUNDMENT
Tributary Channel
Receiving Channel
Epilimnion (Mixed, Flowing)
Thermocline (Essentially Stagnant)
Hypolimnion (Stagnant)
Sediment Layer
(2)
D
T
LARGE SURFACED, MODERATELY SHALLOW IMPOUNDMENT, VERY LOW VELOCITIES
Tributary Channel
Receiving Channel
Epilimnion
Thermocline
Hypolimnion
Sediment Layer
Gradual decrease in depth with large increase in surface area
Mixing decreased
Increased mixing
(3)
D
T
SHALLOW, WIND MIXED (TURBULENT) IMPOUNDMENT
Wind Shear
Tributary Channel
Receiving Channel
Sediment Layer
(4)
D
T
DEEP, SUPERFICIALLY TURBULENT, STRATIFIED IMPOUNDMENT
Wind Shear
Tributary Channel
Receiving Channel
Epilimnion
Thermocline
Hypolimnion
Sediment Layer
(5)
D
T

FIGURE V-13 IMPOUNDMENT CONFIGURATIONS AFFECTING SEDIMENTATION

TABLE V-8

HYPOTHETICAL PHYSICAL CHARACTERISTICS OF LOWER LAKE, BROOKHAVEN, SUFFOLK COUNTY, NEW YORK (ASSUMING AN EPILIMNION DEPTH OF 10 FEET)

Distance Downstream from Inflow Miles (feet)	D Average Depth ft.	W Average Width ft.	CSA Cross-sectional Area, D x W ft^2
0.075	10	160	1,600
0.150	10	170	1,700
0.225	10	175	1,750
0.300	10	200	2,000
0.375	10	198	1,980
0.450	10	230	2,300
0.525	10	233	2,330
0.600	10	200	2,000
0.675	10	222	2,220
0.750	10	316	3,160
0.825	10	435	4,350
0.900	10	590	5,900
0.975	10	552	5,520
1.050	10	435	4,350
1.125	10	325	3,250

Total length = 1.125 mi (5,940 ft.) mean CSA = 2,961 ft^2

Inflow from upstream 397 cfs
Outflow to downstream 393 cfs
(steady-state surface, difference due to loss to water table)

Average flow = 395 cfs

Computation

Volume (Vol) = Total length x mean cross-sectional area

$$\text{Vol} = 5{,}940 \text{ ft.} \times 2{,}961 \text{ ft}^2 = 1.76 \times 10^7$$

Residence Time (τ_w) = Vol/flow

$$\tau_w = 1.76 \times 10^7/(395 \text{ ft}^3/\text{sec}) = 4.46 \times 10^4 \text{ sec } (12.3 \text{ hr})$$

Velocity (Vel) = length/τ_w

$$\text{Vel} = 5{,}940 \text{ ft}/4.46 \times 10^4 \text{ sec} = 0.133 \text{ ft/sec}$$

EXAMPLE V-7

Large, Irregular Surface Impoundment

Figure V-14 shows Kellis Pond and surrounding topography. This small pond is located near Bridgehampton, New York and has a surface area of about 36 acres. From the surface shape of the pond, it is clear that it cannot be considered as a stream reach.

Figure V-15 shows a set of hypothetical depth profiles for the pond. From the profiles, it is evident that considerable shoaling has resulted in the formation of a relatively well defined flow channel through the pond. Peripheral stagnant areas have also formed. Hypothetical velocity vectors for the pond are presented in Figure V-16. Based upon them, it is reasonable to consider the pond as being essentially the hatched area in Figure V-15. To estimate travel times, the hatched area may be handled in the same way as for the Upper Lake example presented above. It should be noted, however, that this approach will almost certainly result in underestimation of sediment deposition in later computations. This is true for two reasons. First, estimated travel time will be smaller than the true value since impoundment volume is underestimated. Second, since the approach ignores the low flow velocities to either side of the central channel and nonuniform velocities within it, heavier sedimentation than computed is likely.

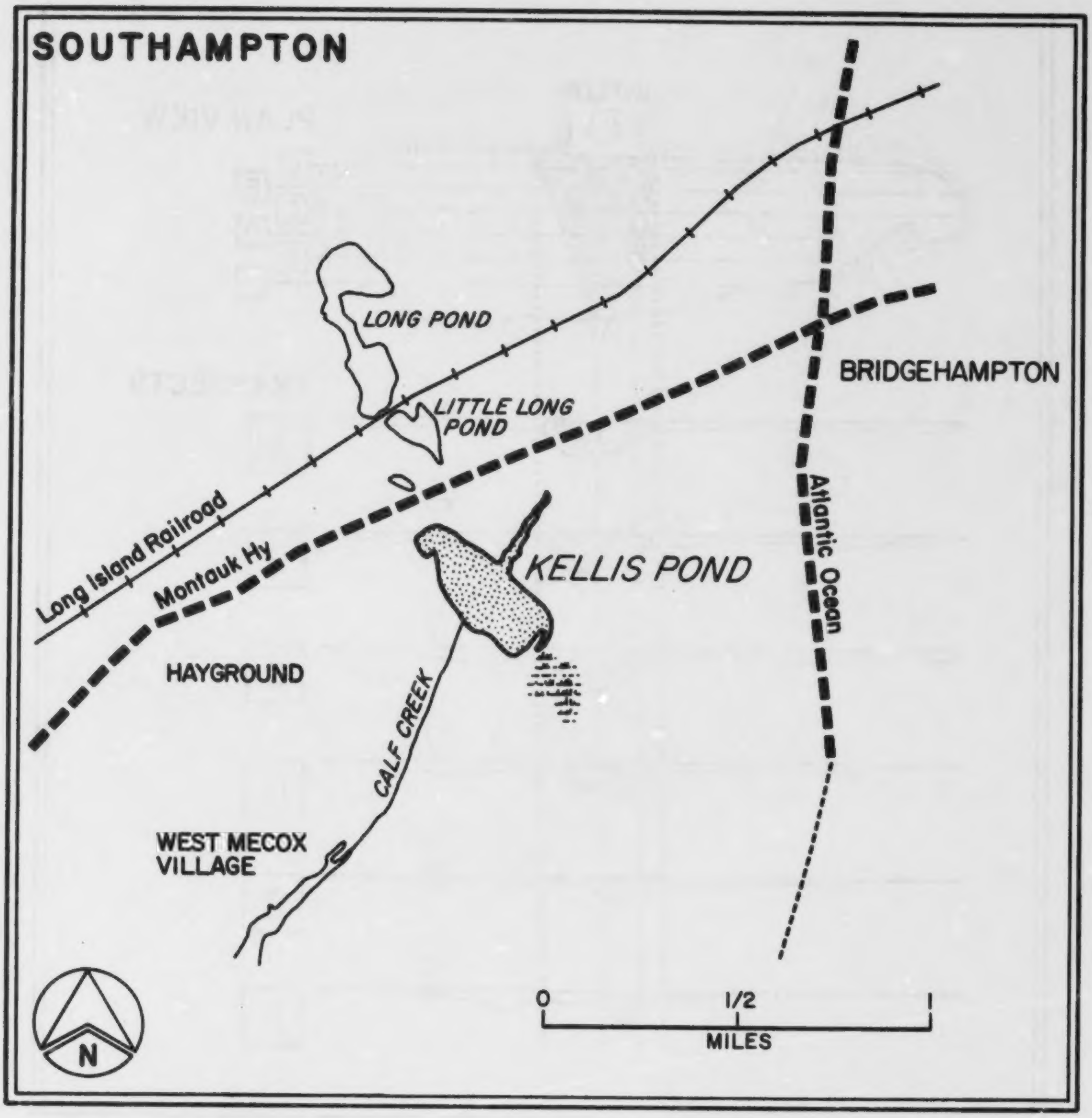

SOUTHAMPTON
LONG POND
LITTLE LONG POND
Long Island Railroad
Montauk Hy
KELLIS POND
BRIDGEHAMPTON
Atlantic Ocean
HAYGROUND
CALF CREEK
WEST MECOX VILLAGE
0
1/2
1
MILES
N

FIGURE V-14 KELLIS POND AND SURROUNDING REGION, LONG ISLAND, NEW YORK

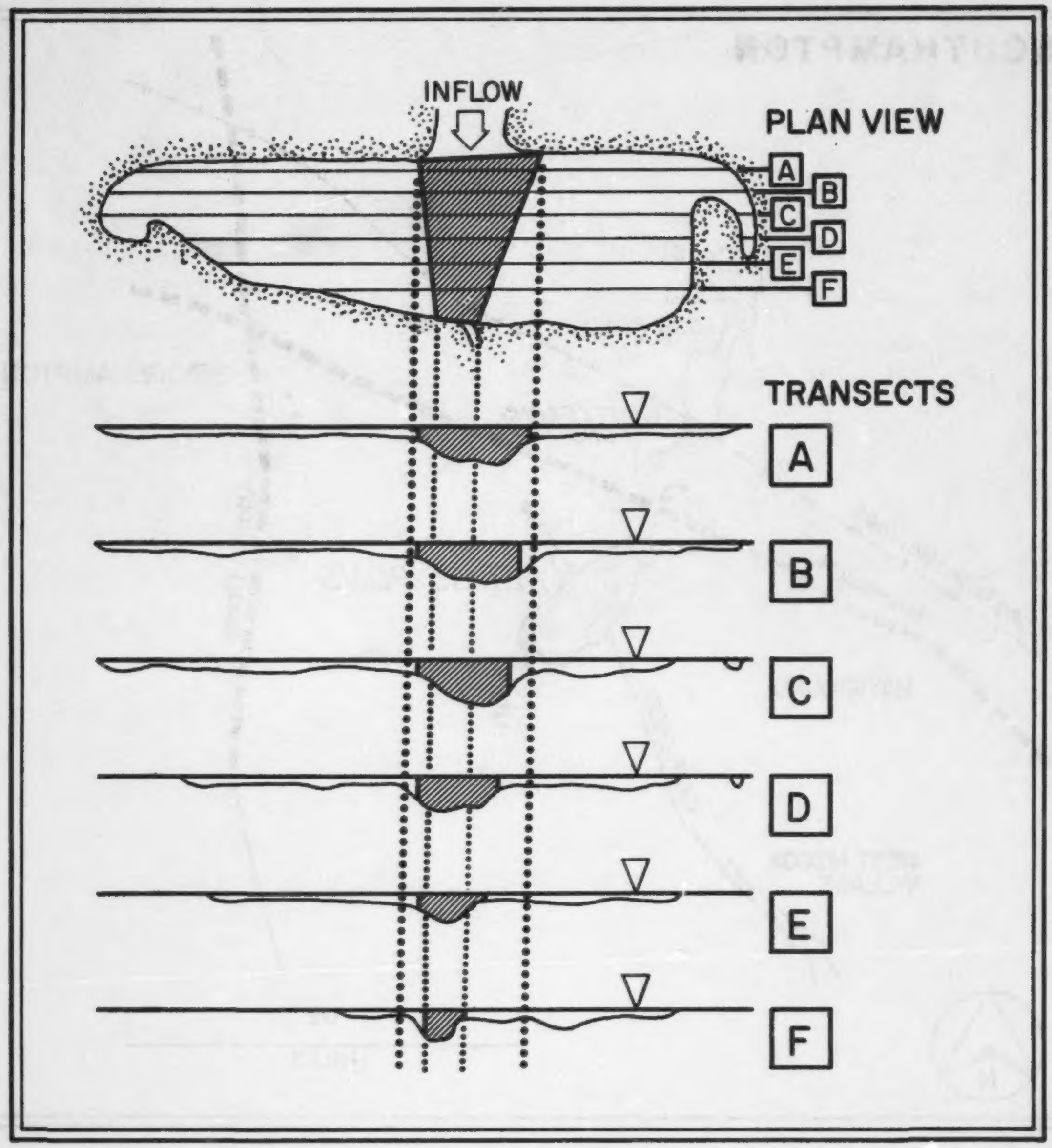

INFLOW
PLAN VIEW
A
B
C
D
E
F
TRANSECTS
A
B
C
D
E
F

FIGURE V-15 HYPOTHETICAL DEPTH PROFILES FOR KELLIS POND

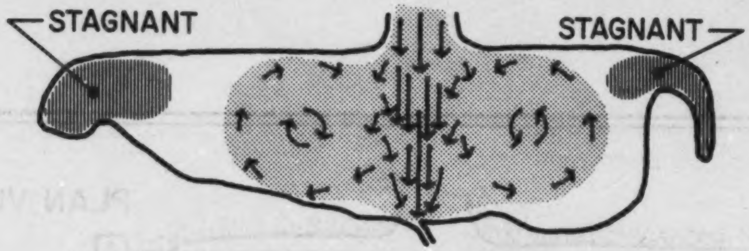


FIGURE V-16 HYPOTHETICAL FLOW PATTERN IN KELLIS POND

Still more difficult to evaluate is the situation where shoaling and scour have not resulted in formation of a distinct central channel. Figure V-17 shows hypothetical depth profiles for Kellis Pond for such a case.

Here, velocity distribution data should be obtained, particularly if the impoundment is of much importance. If such data are not available but it is deemed worthwhile to do field studies, methods available for evaluating flow patterns include dye tracing and drogue floats. A simple but adequate method (at least to evaluate the surface velocity distribution) is to pour a large number of citrus fruits (oranges, grapefruit) which float just below the surface, into the impoundment, and to monitor both their paths and velocities. Although it is true that surface velocities may be greater than the velocity averaged over depth, this will permit estimation of hydraulic residence time directly or generation of data to use in the prescribed method. In the latter case, the data might be used to define the major flow path through an impoundment of a form like Kellis Pond.

———————— END OF EXAMPLE V-7 ————————

———————— EXAMPLE V-8 ————————

Complex Geometries

The final hydraulic residence time example shows the degree of complexity that sediment deposition problems may entail. Although it is possible to make rough estimates of sediment accumulation, it

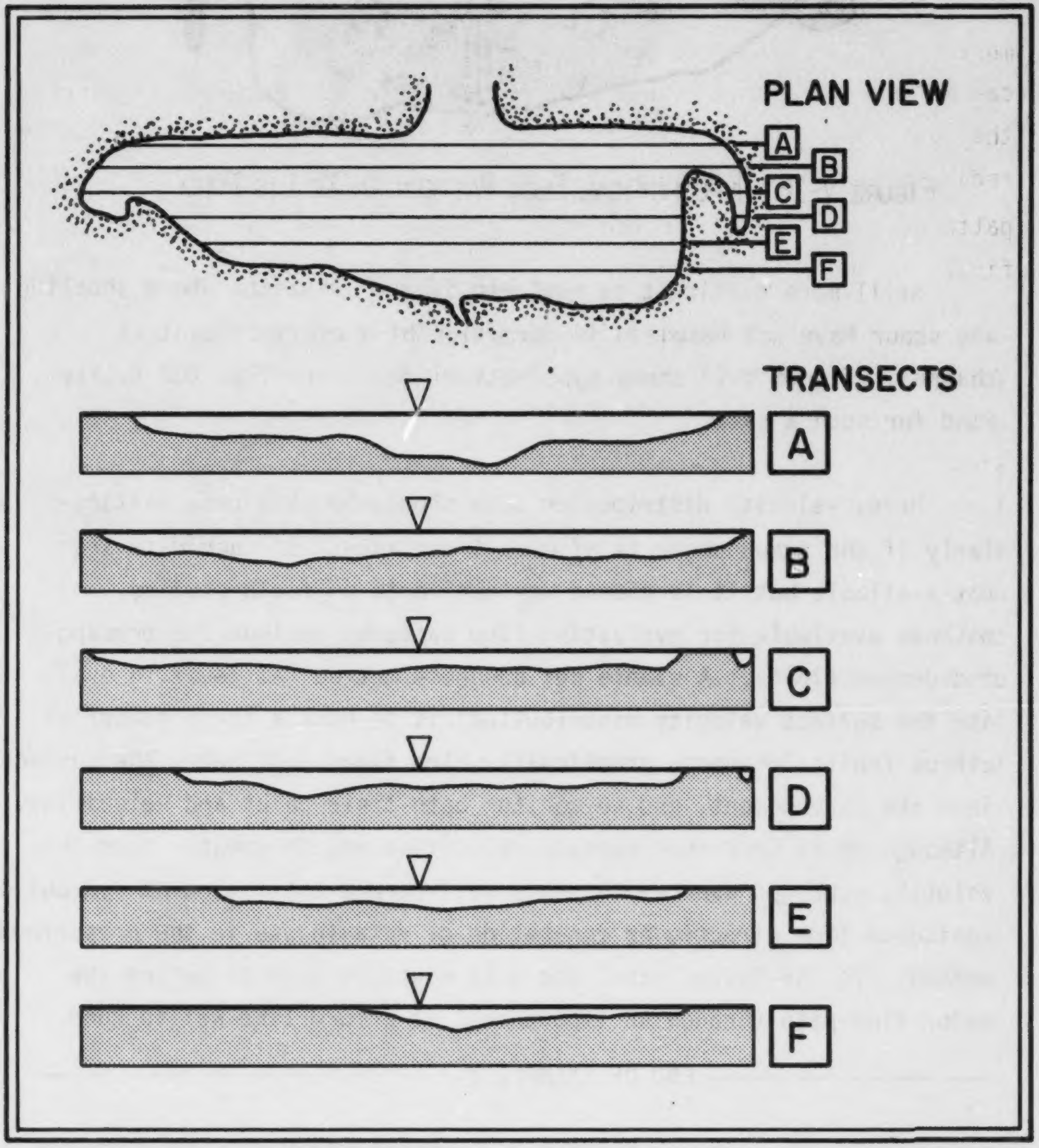

PLAN VIEW
A
B
C
D
E
F
TRANSECTS
A
B
C
D
E
F

FIGURE V-17 HYPOTHETICAL DEPTH PROFILES FOR KELLIS POND NOT SHOWING SIGNIFICANT SHOALING

is recommended that for such impoundments more rigorous methods be used - mathematical modeling and/or detailed field investigations.

Figure V-18 shows Lake Owyhee in eastern Oregon. This impoundment is well outside the range of complexity of water bodies which can be evaluated using these calculation methods. Because of geometry, the number of tributaries, and size, it isn't feasible to conceptually reduce the impoundment in such a way as to estimate travel times. Flow patterns are likely to be very complex, and sediment deposition is difficult to predict both in terms of quantity and location.

In contrast, Figure V-19 shows New Millpond near Islip, New York and surrounding features. Although this water body does not have a simple surface geometry, it can be reduced to three relatively simple components as shown in the figure. Bearing in mind the limitations imposed by wind mixing, stratification, and the presence of stagnant regions described in earlier examples, deposition might nevertheless be estimated in arms A, B, and C. Because of the difficulty of predicting velocities and turbulence in section D, estimates of sedimentation cannot be reliably made there. However, it is likely that much of inflowing sediments will have settled out by the time water flows through the arms and into section D.

END OF EXAMPLE V-8

5.3.5 Estimation of Sediment Accumulation

Estimation of quantities of sediment retained in an impoundment follows directly from the computations of settling velocity and travel time, although the computation depends upon whether the adopted model is plug flow, or a fully mixed layer or impoundment.

In the case of plug flow, one of two subordinate assumptions is made: that the plug is fully mixed as in turbulent flow, or that it moves in a "laminar" flow through the impoundment. In terms of sediment accumulation estimates, the fully mixed plug assumption is

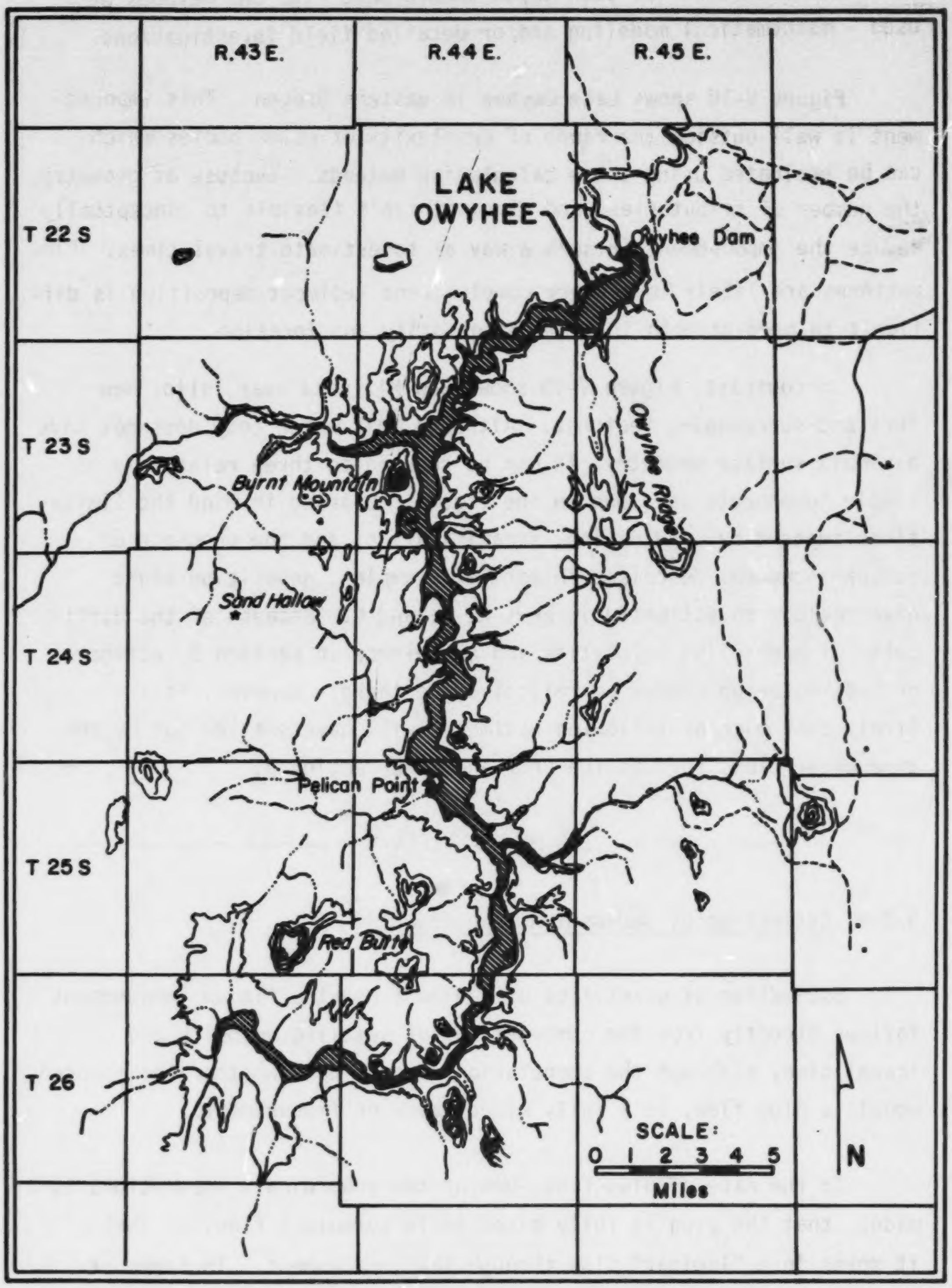

R.43 E.
R.44 E.
R.45 E.
LAKE OWYHEE
Owyhee Dam
T 22 S
T 23 S
T 24 S
T 25 S
T 26
Owyhee Ridge
Burnt Mountain
Sand Hallow
Pelican Point
Red Butte
SCALE:
0 1 2 3 4 5
Miles
N

FIGURE V-18 LAKE OWYHEE AND ENVIRONS

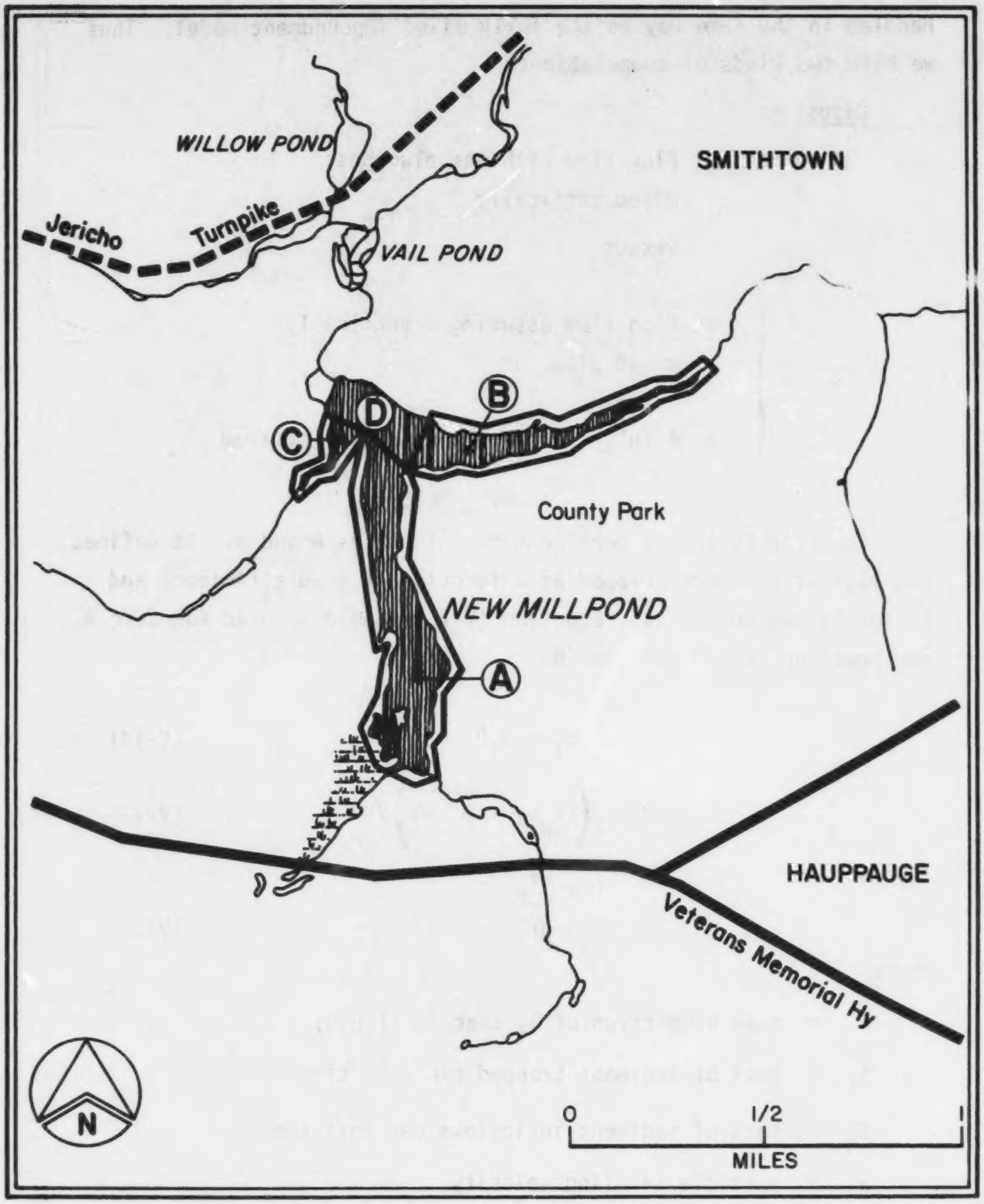

WILLOW POND
SMITHTOWN
Jericho
Turnpike
VAIL POND
B
D
C
County Park
NEW MILLPOND
A
HAUPPAUGE
Veterans Memorial Hy
N
0
1/2
1
MILES

FIGURE V-19 NEW MILLPOND AND ENVIRONS. NEW MILLPOND IS SUBDIVIDED FOR PURPOSES OF ESTIMATING SEDIMENTATION IN REGIONS A, B, AND C.

handled in the same way as the fully mixed impoundment model. Thus we have two kinds of computations:

Cases

A
- Plug flow with the plug not mixed vertically

versus

B
- Plug flow assuming a vertically mixed plug, or
- A fully mixed impoundment or stratum

Equation (V-12) is pertinent to both cases A and B. It defines the mass of sediment trapped as a function of trap efficiency and inflowing sediment mass. Equation (V-13) should be used for case A, and Equation (V-14) for case B.

$$S_t = S_i P \qquad \text{(V-12)}$$

$$P = \Big((\tau_w v) + D'' - D\Big)/D'' \qquad \text{(V-13)}$$

$$P = \frac{v\tau_w}{D'} \qquad \text{(V-14)}$$

where

P = mean proportion of S_i trapped ($1 \geq P \geq 0$)

S_t = mass of sediment trapped per unit time

S_i = mass of sediment in inflows per unit time

v = particle settling velocity

D = discharge channel depth

D' = flowing layer depth

D'' = inflow channel depth

Figure V-20 shows the significance of the various depth measures D, D', and D'', and the assumed sedimentation pattern. In case B, in the absence of substantial erratic turbulence and unpredicted vertical velocity components, and within the constraints of available data, it is clear that this approach can give reasonable estimates of trap efficiencies. In case A, however, small changes in D or D'' can strongly affect trap efficiencies. It is important to remember in applying case A that P is a mean, preferably used over a period of time. It is also important to recognize that conditions within an impoundment leading to selection of case A or B are subject to change, thus affecting estimates.

For convenience, Figure V-21 is included to provide estimates of v_{max} for spherical particles of 2.7 specific gravity. The data are presented as a function of particle diameter and temperature. Figure V-22 is a nomograph relating trap efficiency, P (in percent) to depth D', v_{max}, and τ_w. The nomograph is useful only for case B assumptions.

EXAMPLE V-9

Sedimentation in Upper and Lower Lakes

Using the data from Table V-6 and settling velocities for the clay and sand of Example V-4, for case A,

$$\tau_w = 1.6\text{x}10^4 \text{ sec}$$

$$v_{max} \text{ for clay} = 8 \text{ ft day}^{-1}$$

$$v_{max} \text{ for sand} = 252 \text{ ft hour}^{-1}$$

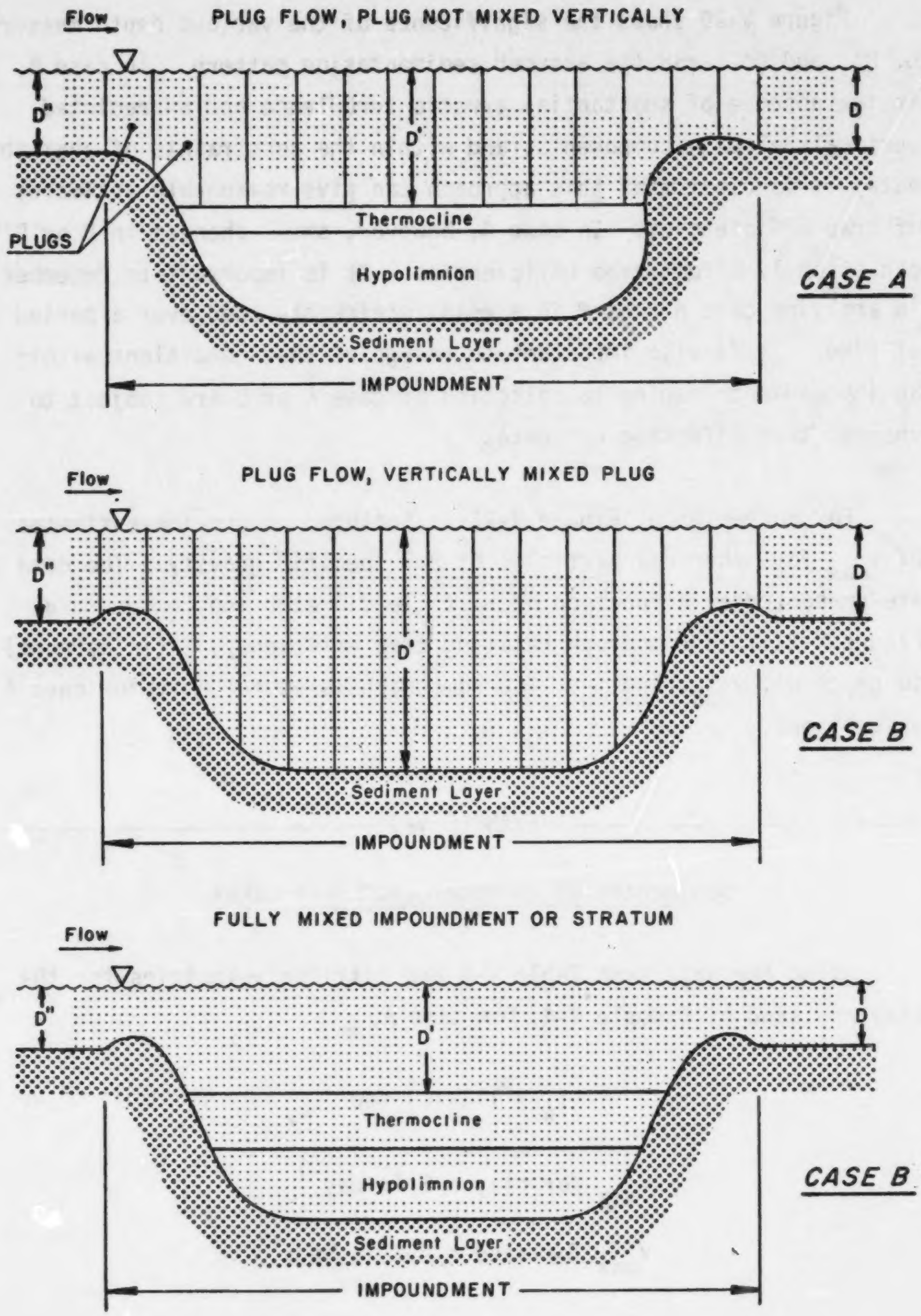

Flow
PLUG FLOW, PLUG NOT MIXED VERTICALLY
D"
D'
D
PLUGS
Thermocline
Hypolimnion
CASE A
Sediment Layer
IMPOUNDMENT
Flow
PLUG FLOW, VERTICALLY MIXED PLUG
D"
D'
D
CASE B
Sediment Layer
IMPOUNDMENT
Flow
FULLY MIXED IMPOUNDMENT OR STRATUM
D"
D'
D
Thermocline
Hypolimnion
CASE B
Sediment Layer
IMPOUNDMENT

FIGURE 20 SIGNIFICANCE OF DEPTH MEASURES D, D^1, AND D^{11}, AND THE ASSUMED SEDIMENTATION PATTERN

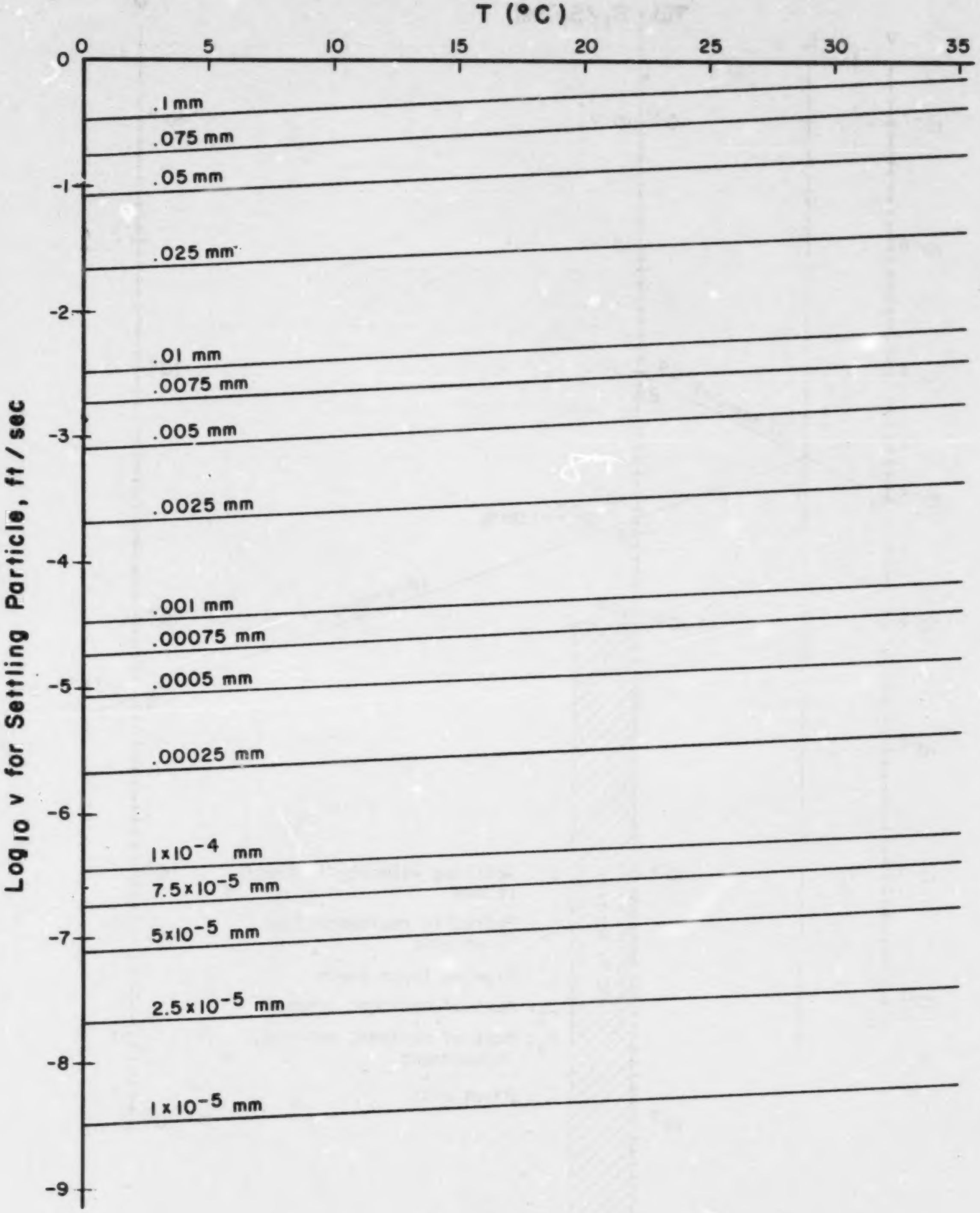


FIGURE V-21 SETTLING VELOCITY FOR SPHERICAL PARTICLES

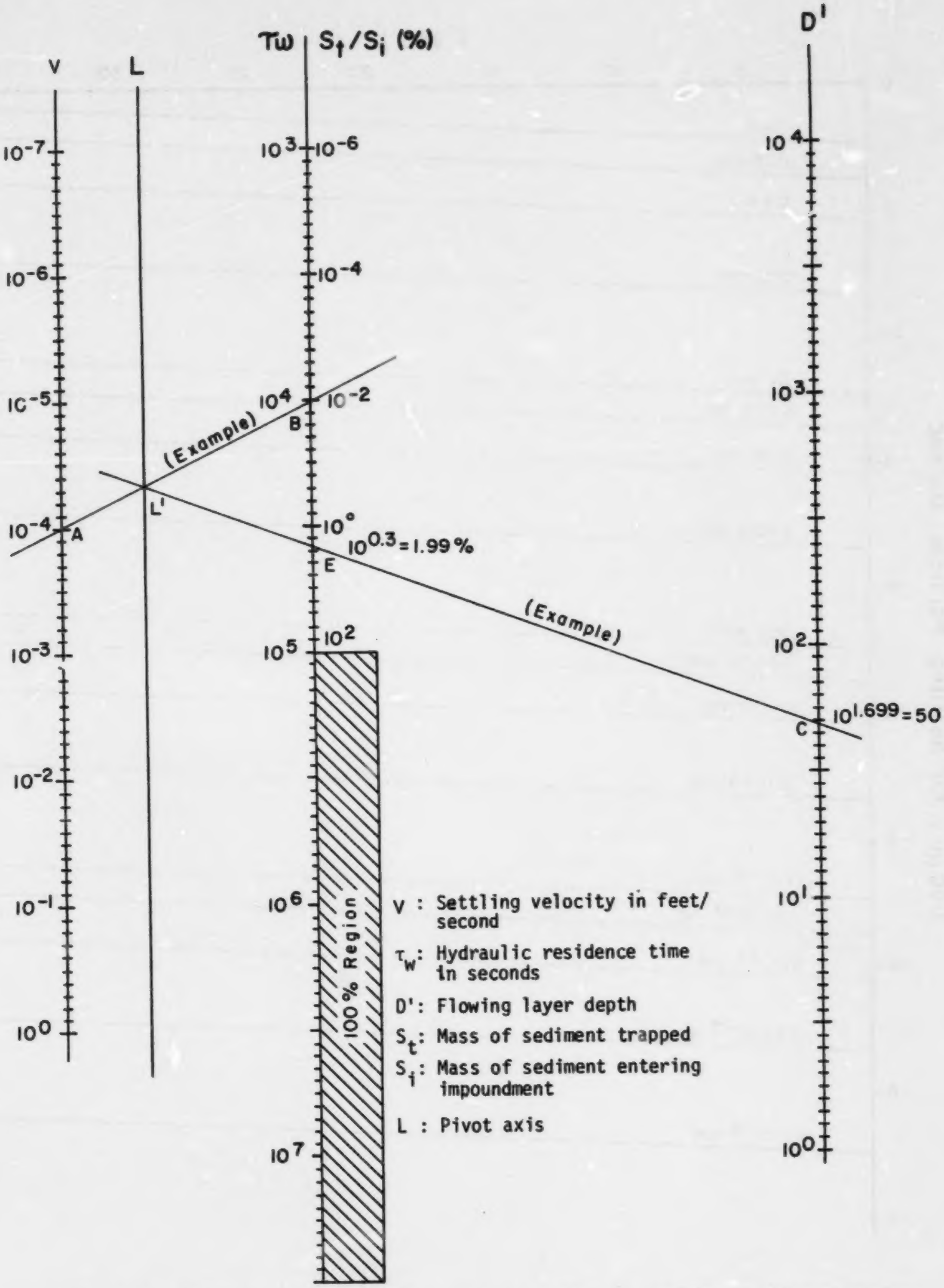


FIGURE V-22 NOMOGRAPH FOR ESTIMATING SEDIMENT TRAP EFFICIENCY

Although it is not specified in Table V-6, the inflow channel depth at the entrance to Upper Lake is 3 feet. The discharge channel depth is 10 feet. Assuming "laminar" flow with minimal vertical components, for clay:

$$\tau_w = \frac{[(T_w \times v) + D'' - D]}{D''}$$

$$P = \frac{[(1.6\text{x}10^4\text{x}9.3\text{x}10^{-5}) + 3 - 10]}{3}$$

$$P = -5.5$$

The negative value implies that the proportion settling out is virtually zero. Thus the clay will to a large extent pass through Upper Lake. However, τ_w for this example is very small (4.5 hours). Many impoundments will have substantially larger values.

For the sand,

$$P = \frac{[(1.6\text{x}10^4\text{x}7\text{x}10^{-2}) + 3 - 10]}{3}$$

$$P = 371$$

All of the sand will clearly be retained. Note that a clay or very fine silt of $v_{max} = 5\text{x}10^{-4}$ ft $\sec^{-1}$ would be only partially trapped.

$$P = \frac{[(1.6\text{x}10^4\text{x}5\text{x}10^{-4}) + 3 - 10]}{3}$$

$$P = 0.33$$

Thus about one-third of this sediment loading would be retained. Note that if D is large, trap efficiency drops using this algorithm. For the silt, a discharge channel depth (at the outflow from Upper Lake) of 11 feet rather than 10 would give

$$P = \frac{[(1.6 \times 10^{4} \times 5 \times 10^{-4}) + 3 - 11]}{3} = 0$$

Thus with D=11, all silt exits the impoundment. If D is only 9 feet, then

$$P = \frac{[(1.6 \times 10^{4} \times 5 \times 10^{-4}) + 3 - 9]}{3} = .66$$

Two-thirds of the silt is retained. Remember that P represents a mean value. Clearly during some periods none of the silt will be retained (due to turbulence, higher velocities) while during other periods, all of the silt will be trapped. The key here is the word "mean."

If the impoundment is assumed to be vertically mixed (case B), compute the mean depth $\bar{D}$

$$\bar{D} = \sum_{i=1}^{n} D_i/n$$

where

n = the number of cross-sections

D_i = depth at the ith cross-section

For Upper Lake,

$$\bar{D} = 6.7 = D'$$

Then

$$P = \frac{v\,\tau_w}{D'}$$

For the clay,

$$P = \frac{9.3\times10^{-5}\times1.6\times10^{4}}{6.7} = 0.22$$

About one-fourth of the clay is retained.

For the sand,

$$P = \frac{7\times10^{-2}\times1.6\times10^{4}}{6.7} = 167$$

All of the sand will be trapped within about 1/167 times the length of the lake. If the daily influent loading of sand is one ton, while the loading of clay is fifteen tons, then the daily accumulation will be one ton of sand and 0.22 x 15 = 3.3 tons of clay.

Finally, as an example of use of Figures V-21 and V-22, assume that the sediment loading consists primarily of silt particles in the size range of .002mm diameter, and that the water temperature is 5^oC. Further, assume τ_w has been estimated as 2.77 days (10^4 seconds), and that D'=50 feet. From Figure V-21, the settling velocity is about 1×10^{-4} feet per second.

In Figure V-22, draw a line from 10^{-4} on the V axis to 10^4 on the τ_w axis. The point of intersection with axis L is L'. Next, compute $\log_{10}50=1.699$. Draw a line from this point on the D' axis to L'. Where this line crosses the S_t/S_i (%) axis gives the log of the percent of the sediment trapped. This is $10^{0.3}=1.99\approx2\%$.

---------------------------- END OF EXAMPLE V-9 ----------------------------

5.4 EUTROPHICATION

5.4.1 Introduction

The presence of nutrients in an impoundment generally favors plant growth. Depending upon antecedent conditions, the relative abundance of nitrogen, phosphorus, light, and heat, and the status of a number of other physical and chemical variables, the predominant forms may be diatoms, microscopic or macroscopic algae, or bottom-rooted or free-floating macrophytes. The quantity of plant matter present in an impoundment is important for several reasons. First, plant cells produce oxygen during the "light reaction" of photosynthesis, thereby providing an important source of dissolved oxygen to the water column. Plant cells also consume oxygen through the process of respiration. Respiration occurs along with photosynthesis during the day, but also occurs at night. Oxygen consumed at night may be considerable, not only because it serves to sustain the plant cells, but because the cells actively perform various vital metabolic functions in the dark. One such process, called the "dark reaction", serves to store energy which was intercepted by the plant cells during the "light reaction". The energy is usually stored as the simple monosaccaride glucose or as glucose polymers.

Plant growth within an impoundment is also important because plant biomass is a major source of nutrition for indigenous fauna, and the growth of plants constitutes what is called "primary production." The stored energy and nutrients provide food for various grazers higher in the food chain, either through direct consumption of living plant tissue by fishes and zooplankton or through consumption of detritus by fishes, microorganisms, and zooplankton. The grazers, in turn, provide food for predatory fishes, mammals, insects, and other higher forms.

Finally, plant development in impoundments is important because it tends to accelerate impoundment aging. As plants grow, organic matter and sediment accumulate. As the impoundment fills with rock fragments, soil, and plant detritus, an excellent substrate forms upon which more suspended matter may be trapped and which may ultimately support the growth of higher plants and trees. The gradual filling in of an impoundment in this way reduces its usefulness, and may finally eliminate the impoundment completely.

5.4.2 Nutrients, Eutrophy, and Algal Growth

Eutrophy means literally a state of good nutrition. Plants require a number of nutrients, but to vastly different degrees. Some nutrients, such as nitrogen, potassium, and phosphorus, are needed in large quantity. These are termed macronutrients. The micronutrients, e.g. iron, cobalt, manganese, zinc, and copper, are needed in very small amounts. In nature, the micronutrients are usually in adequate supply (although not always), while nitrogen and phosphorus are commonly growth limiting.

Nitrogen, particularly as nitrate and ammonium ions, is available to water-borne plant cells to be used in synthesis of proteins, chlorophyll a, and plant hormones. Each of these substances is vital for plant survival.

Phosphorus, an element found in a number of metabolic cofactors, is also necessary for plant nutrition. The biosynthesis and functioning of various biochemical cofactors rely on the availability of phosphorus, and these cofactors lie at the very foundation of plant cell metabolism. Without adequate phosphorus, plant cells cannot metabolize properly.

Since nitrogen and phosphorus are commonly in limited supply, many impoundments tend inherently to be clear and essentially free of clogging algae and vascular plants. Because of society's ever-increasing size and need for food, chemical sources of nitrogen and phosphorus are synthesized and spread over vast tracts of farmland. Stormwater washes off these nutrients, which then flow through streams and into natural and artificial impoundments. Due to the fact that many impoundments have very low flow velocities, impoundments represent excellent biological culturing vessels, and often become choked with plant life.

Since a plant cell has, at any point in time, a specific need for nitrogen and for phosphorus, one or the other or both may limit cell growth or replication. Where nitrogen is the nutrient that restricts the rate of plant growth, that is, where all other nutrients and factors are present in excess, we say that nitrogen is growth limiting. In general, N:P ratios in the range of 5 to 10 by mass are usually associated with plant growth being neither nitrogen nor phosphorus limited. However, in this range, plant growth may well be limited by N and P collectively. Where the ratio is greater than 10, phosphorus tends to be limiting, and for ratios below 5, nitrogen tends to be limiting. (Chiandani, et al., 1974).

In addition to nitrogen and phosphorus, any necessary nutrient or physical condition may limit plant growth. For example, in highly nutritious (euthophic) waters, algal biomass may increase until light cannot penetrate, and light is then limiting. At such point, a dynamic equilibrium exists in which algal cells are consumed, settle, or lyse (break) at the same rate as new cells are produced.

5.4.3 Predicting Algal Concentrations

Predicting algal blooms or predominance of macrophytes using a mechanistic approach can be a very complex problem, and most methods are not suited to a simple hand calculations technique. Some relationships regarding algal productivity have been derived, however, which permit an evaluation of the eutrophic state of an impoundment. Because the methods permit algal biomass to be estimated with relatively little, easily obtained data, and because algae are very important in assessing impoundment water quality, these techniques are useful here.

The methods presented below are based upon the fact that in most cases (perhaps 60 percent) phosphorus is the biomass limiting nutrient (EPA, 1972). One such approach has been developed by Vollenweider (EPA, 1972, Lorenzen, 1976). It may be used to predict the degree of impoundment eutrophication as a function of areal phosphorus loading. It does not, however, permit direct estimates of algal biomass to be made. Figure V-23 shows the relationship as a plot of phosphorus loading (L_p) versus the ratio of impoundment mean depth ($\bar{Z}$) to hydraulic residence time (τ_w). As indicated in the plot, it is possible to predict on a preliminary basis, whether an impoundment will have eutrophication problems. Figure V-24 shows where 133 eastern U.S. impoundments are located on the same kind of plot.

Before considering application of any of the methods to assess eutrophication, it is important to examine the nitrogen to phosphorus ratio. This indicates whether any of the methods presented below is likely to give realistic results.

EXAMPLE V-10

Big Reservoir and The Vollenweider Relationship

To use the Vollenweider relationship for phosphorus loading, data on long-term phosphorus and nitrogen loading rates must be available. It is also important that the rates represent average loading conditions over time because transient phosphorus loading pulses will give misleading results. Big Reservoir has the following characteristics:

Big Reservoir

Available Data (all values are means):

Length	20 mi	= 32.2 km
Width	5 mi	= 8.05 km

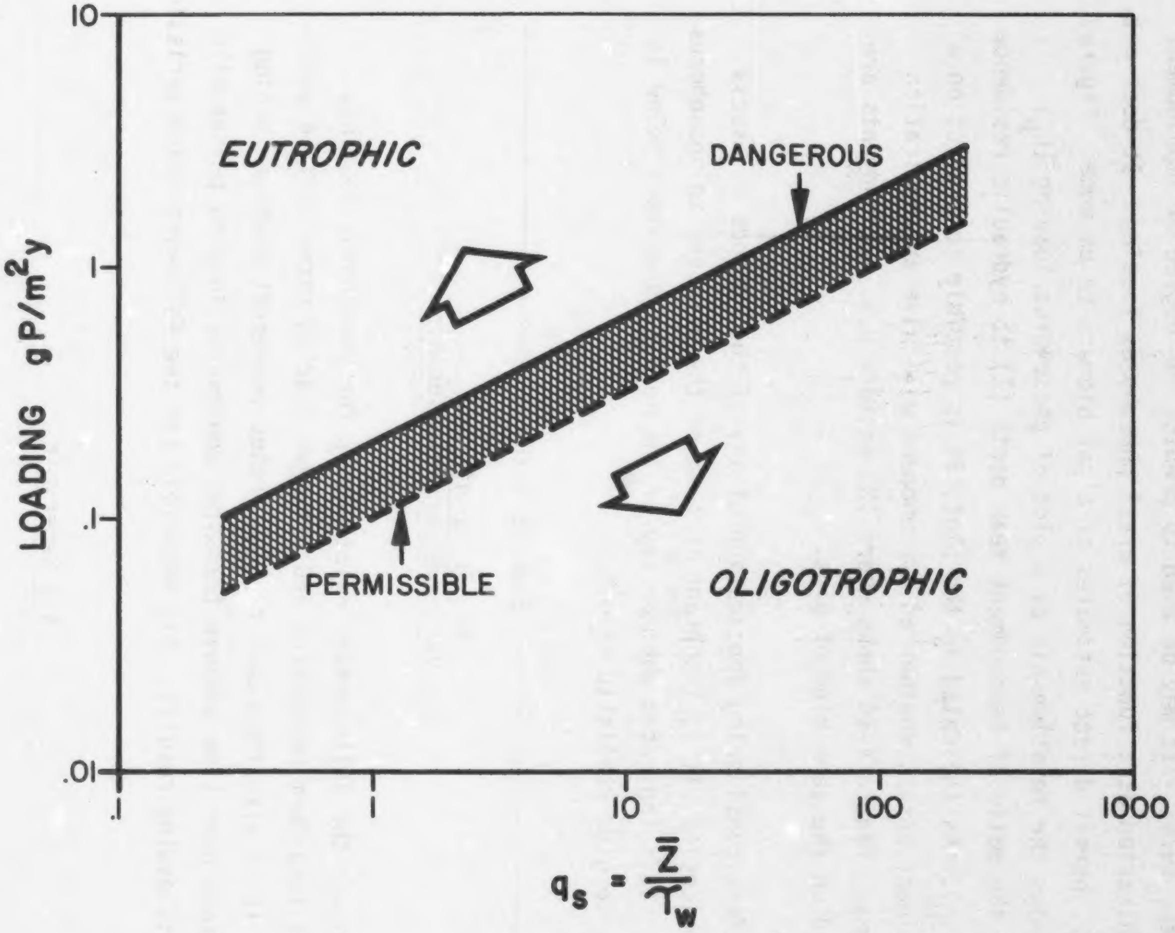


FIGURE V-23 THE VOLLENWEIDER RELATIONSHIP

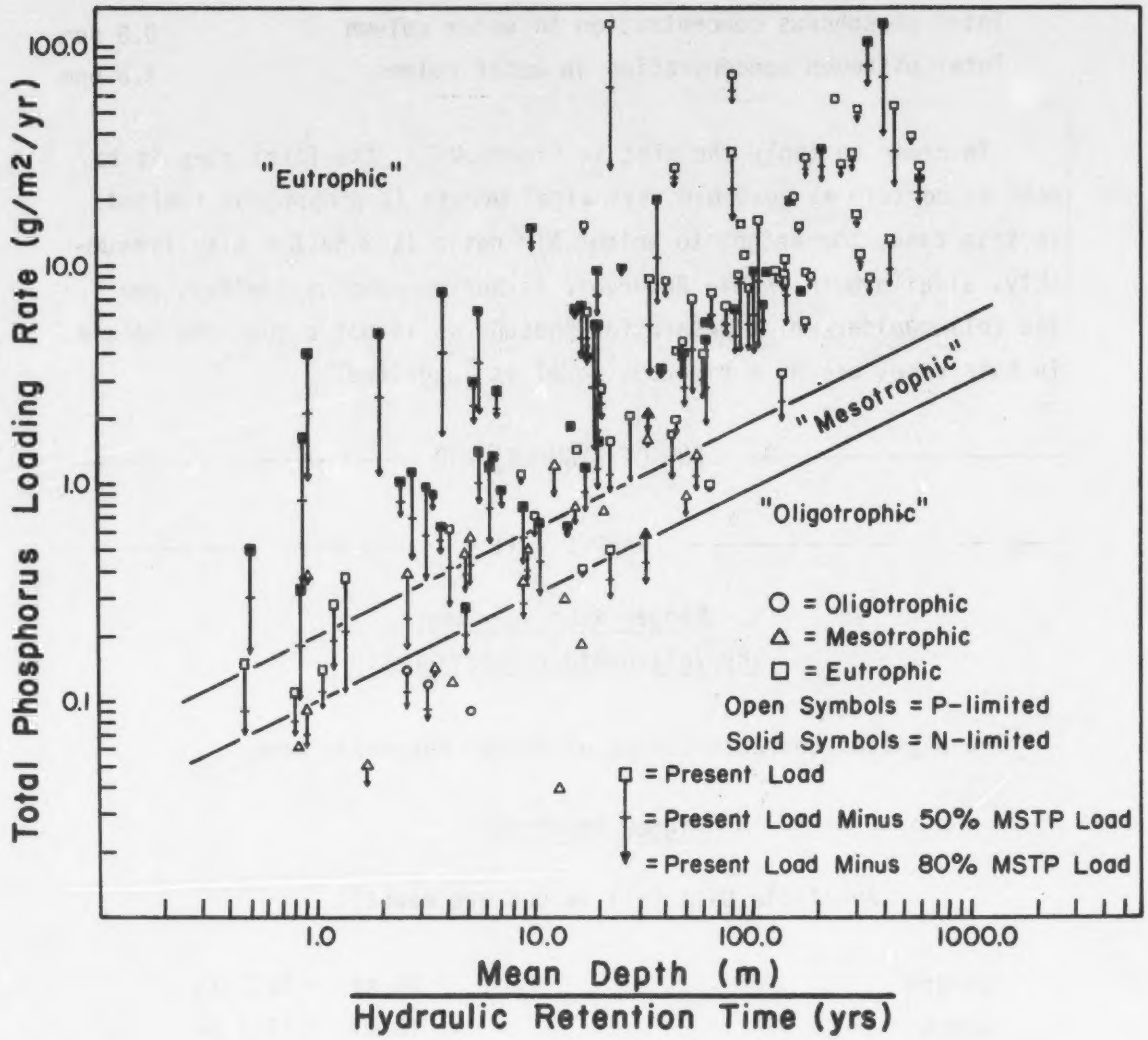


FIGURE V-24 PLOT OF THE VOLLENWEIDER RELATIONSHIP SHOWING THE POSITION OF 133 EASTERN U.S. IMPOUNDMENTS IMPACTED BY MUNICIPAL SEWAGE TREATMENT PLANT (MSTP) EFFLUENTS (NATIONAL EUTROPHICATION SURVEY, 1976)

Depth ($\bar{Z}$)	200 ft = 61 m	
Inflow (Q)	500 cfs	
Total phosphorus concentration in water column		0.8 ppm
Total nitrogen concentration in water column		3.6 ppm

In order to apply the plot in Figure V-23, the first step is to make as certain as possible that algal growth is phosphorus limited. In this case, the weight to weight N:P ratio is 3.6/.8 = 4.5. Presumably, algal growth in Big Reservoir is not phosphorus limited, and the Vollenweider relationship for phosphorus is not a good one to use. In this case, use of a rigorous model is suggested.

END OF EXAMPLE V-10

EXAMPLE V-11

Bigger Reservoir and The Vollenweider Relationship

The physical characteristics of Bigger Reservoir are:

Bigger Reservoir

Available Data (all values are means):

Length	20 mi	= 32.2 km
Width	10 mi	= 16.1 km
Depth ($\bar{Z}$)	200 ft	= 61 m
Inflow (Q)	500 cfs	
Total phosphorus concentration in inflow		0.8 ppm
Total nitrogen concentration in inflow		10.6 ppm

As in the preceding example, first determine whether phosphorus is likely to be growth limiting. Since data are available only for influent water, and since no additional data are available on impoundment water quality, N:P for influent water will be used.

$$N:P = 10.6/0.8 = 13.25$$

Thus algal growth in Bigger Reservoir is probably phosphorus limited.

Compute the approximate surface area, volume and the hydraulic residence time (for technique, see sediment section above).

$$\text{Volume } (V) = 20 \text{ mi} \times 10 \text{ mi} \times 200 \text{ ft} \times 5280^2 = 1.12 \times 10^{12} \text{ft}^3 = 3.16 \times 10^{10} \text{m}^3$$

$$\text{Hydraulic residence time } (\tau_w) = V/Q = 1.12 \times 10^{12} \text{ft}^3/500 \text{ ft}^3\text{sec}^{-1} = 2.24 \times 10^9 \text{sec} = 71 \text{ yr}$$

$$\text{Surface area } (A) = 20 \text{ mi} \times 10 \text{ mi} \times 5280^2 = 5.57 \times 10^9 \text{ft}^2 = 5.18 \times 10^8 \text{M}^2$$

Next, compute q_s

$$q_s = \bar{Z}/\tau_w$$

$$q_s = 61 \text{ m}/71 \text{ yr} = 0.86 \text{ m yr}^{-1}$$

Compute annual inflow, Q_y

$$Q_y = Q \times 3.15 \times 10^7 \text{ sec yr}^{-1}$$

$$Q_y = 1.58 \times 10^{10} \text{ft}^3 \text{ yr}^{-1}$$

Phosphorus concentration in the inflow is 0.8 ppm, or 0.8 mg/l. Loading (L_p) in grams per square meter per year is computed from the phosphorus concentration, in mg/l:

$$L_p = \frac{28.31\ell}{\text{ft}^3} \times \frac{1 \text{ g}}{1000\text{mg}} \times \frac{0.8 \text{ mg}}{\ell} \times \frac{1}{5.18\text{x}10^8 \text{M}^2} \times 1.58\text{x}10^{10} \frac{\text{ft}^3}{\text{yr}}$$

$$L_p = 0.70 \text{ gm}^{-2}\text{yr}^{-1}$$

Now, referring to the plot in Figure V-23, we would expect that Bigger Reservoir is eutrophic, possibly with severe summer algal blooms.

END OF EXAMPLE V-11

EXAMPLE V-12

The Vollenweider Relationship Using Monthly Inflow Quality Data

Is Frog Lake eutrophic? Frog Lake's physical characteristics are as shown below:

Frog Lake

Available Data:

Mean length	2 mi
Mean width	1/2 mi
Mean depth	25 ft

Available Inflow Water Quality Data:

Month	Q (monthly mean, cfs) 1972	1974	Total P (mg/l) 1972	1974	Inorganic N (mg/l) 1972	1974
October	50	65	0.1	0.08	7.2	6.0
November	80	90	0.02	0.02	6.3	2.4
December	40	40	0.03	0.04	3.1	1.5
January	-	-	-	-	-	-
February	-	-	-	-	-	-
March	60	58	0.01	0.02	2.0	1.9
April	80	80	0.01	0.01	2.3	0.50
May	75	76	0.04	0.05	0.55	0.52
June	40	70	0.03	0.08	1.20	1.35
July	-	25	-	0.11	-	2.01
August	38	20	0.09	0.04	3.50	1.29
September	38	25	0.06	0.05	2.80	1.00

First. estimate the mean annual flow and the hydraulic residence time. To compute mean annual flow,

$$Q = \left(\sum_{i=1}^{y} \sum_{j=1}^{n_i} Q_{i,j} \right) / \sum_{i=1}^{y} n_i$$

where

$Q_{i,j}$ = the individual flow measurements

y = the number of years of data

n_i = the number of observations per year

$$Q = 1050/19 = 55.3 \text{ cfs} = 1.75 \times 10^9 \text{ ft}^3/\text{yr}$$

Now estimate the volume, surface area, hydraulic residence time, and q_s

$V = 2 \text{ mi} \times 1/2 \text{ mi} \times 25 \text{ ft} \times 5280^2 = 6.97 \times 10^8 \text{ft}^3 = 1.98 \times 10^7 \text{m}^3$

$A = 2 \text{ mi} \times 1/2 \text{ mi} \times 5280^2 = 2.79 \times 10^7 \text{ft}^2 = 2.59 \times 10^6 \text{m}^2$

$\tau_w = V/Q = 6.97 \times 10^8 \text{ft}^3/55.3 \text{ cfs} = 1.26 \times 10^7 \text{sec} = 0.4 \text{ yr}$

$q_s = 25/0.4 = 62.5$

Next, calculate the weighted mean inflow phosphorus and nitrogen concentrations $\bar{P}$ and $\bar{N}$ as follows:

$$\bar{P} \text{ (or } \bar{N}) = \left(\sum_{i=1}^{y} \sum_{j=1}^{n_i} Q_{i,j} \times C_{i,j} \right) / \left(\sum_{i=1}^{y} \sum_{j=1}^{n_i} Q_{i,j} \right)$$

$$\bar{P} = 43.86/1050 = 0.042 \text{ ppm}$$

$$\bar{N} = 2671.92/1050 = 2.54 \text{ ppm}$$

The N:P ratio in the inflows is 60. Therefore if one of the two is growth limiting, it is probably phosphorus. Compute the phosphorus loading, L_p.

$$L_p = \frac{28.31\ell}{ft^3} \times \frac{1\ g}{1000\ mg} \times \frac{0.042\ mg}{\ell} \times \frac{1}{2.59\times10^6 m^2} \times \frac{1.75\times10^9 ft^3}{yr}$$

$$L_p = 0.80$$

Now, referring to the plot in Figure V-23 with $L_p = 0.80$ and $q_s = 62.5$, the impoundment is well into the oligotrophic region.

END OF EXAMPLE V-12

5.4.4 Winter Phosphorus Levels in Predicting Algal Productivity and Biomass

Another technique, which is also based upon phosphorus loading, may be even more useful than the Vollenweider relationship because it permits summer chlorophyll a concentrations to be estimated rather than general impoundment trophic status. The method has been advanced by several researchers including Sakamoto (1966), Lund (1971), Dillon (1974), and Dillon, et al. (1975). Briefly, the method relates mean summer chlorophyll a concentrations to spring mean total phosphorus. As shown in Figure V-25, the relationship is highly correlated, and a regression of the log of summer mean chlorophyll a on the log of spring mean phosphorus is linear. Using a least squares method gives the equation of the line as (Lorenzen, in press):

$$\log(\text{chl } \underline{a}) = 1.5 \log(P) - 1.1 \qquad \text{(V-15)}$$

or

$$\text{chl } \underline{a} = 0.08(P)^{1.5} \quad P \leq 250\ mg/m^3 = 0.25\ ppm \qquad \text{(V-16)}$$

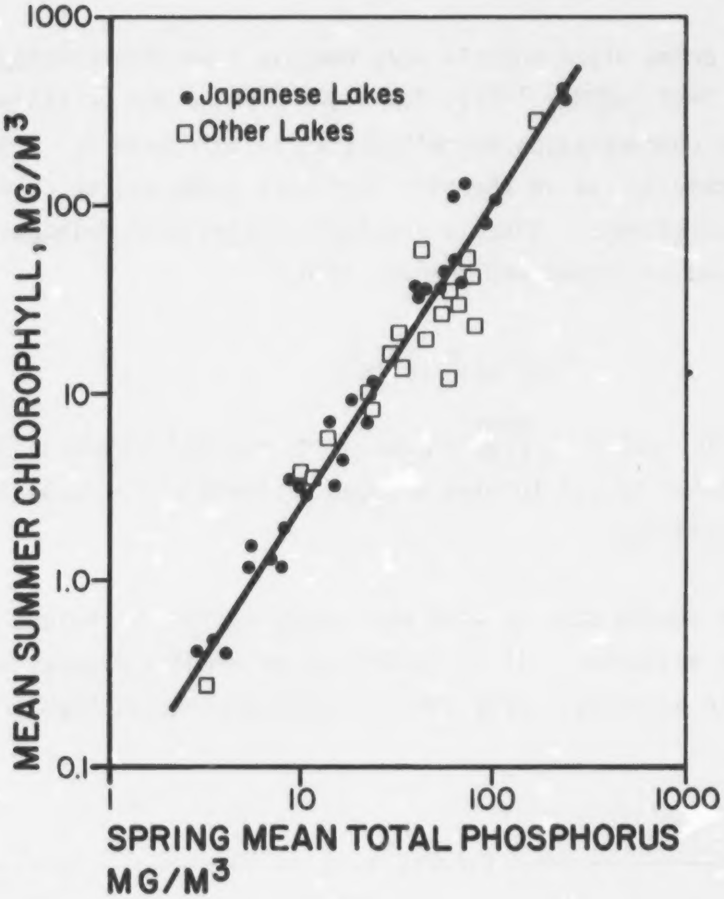


FIGURE V-25 RELATIONSHIP BETWEEN SUMMER CHLOROPHYLL AND SPRING PHOSPHORUS (FROM LORENZEN, UNPUBLISHED)

Figure V-26 shows a plot of maximal primary production in terms of milligrams carbon incorporated in algae per square meter per day as a function of phosphate phosphorus levels. As was the case with predicting chlorophyll a concentrations, the relationship appears to be reasonably robust and therefore useful.

Because dried algae contain very roughly 3 percent chlorophyll a (J.A. Elder, pers. comm., 1977), dry algal biomass may be estimated from chlorophyll a concentration by multiplying by thirty-three. Similarly, carbon productivity, as in the plot in Figure V-26, may be converted to total algal biomass. Since a proximate analysis of dried algae has been determined as (Stumm and Morgan, 1970):

$$C_{106}H_{263}O_{110}N_{16}P_1$$

the gravimetric factor is $\frac{3550}{1272} \simeq 2.8$. Thus, maximal carbon productivity may be multiplied by 2.8 to give a rough estimate of maximal algal biomass productivity.

The user should bear in mind that applying this technique can only lead to rough estimates. If it is desired to predict biomass or productivity with accuracy, more sophisticated approaches may be necessary.

EXAMPLE V-13

Winter Phosphorus and Summer Chlorophyll a

Lake Sara mean spring total phosphorus concentration = .03 mg/l = 30 mg/m^3

$$\text{chl } a = 0.08(P)^{1.5}$$

$$\text{chl } a = 13.1 \text{ mg/m}^3$$

$$\text{algal dry biomass} \simeq 13.1 \times 33 = 430 \text{ mg/m}^3$$

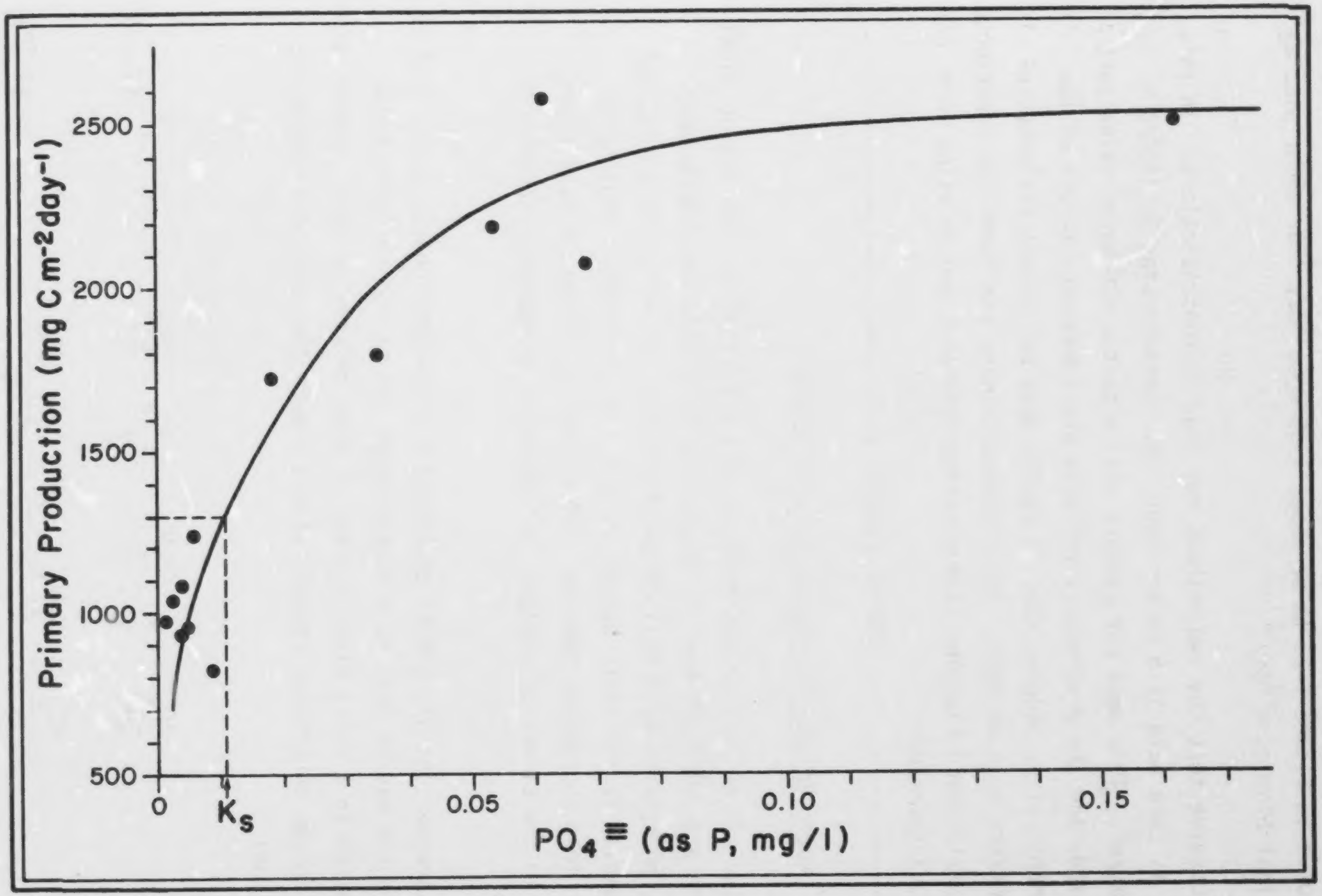


FIGURE V-26 MAXIMAL PRIMARY PRODUCTIVITY AS A FUNCTION OF PHOSPHATE CONCENTRATION (AFTER CHIANDANI, ET AL., 1974)

Maximal carbon productivity in the impoundment may be estimated from the curve in Figure V-26 to be about 1950 $mgCm^{-2}day^{-1}$ or about 5460 mg dry algal biomass $m^{-2}day^{-1}$.

Observe that the two methods may lead to contradictions. In this case, if Lake Sara is 5 meters deep, the concentration is 5460/5 = 1092 mg/m^3. This does not compare well with the 430 mg/m^3 value just computed, and the discrepancy reflects one inadequacy in use of the Chiandani curve, namely, that it really does not permit estimates of concentration to be made. The discrepancy also reaffirms the importance of applying good judgment in evaluating estimates and in using more than one technique.

——————— END OF EXAMPLE V-13 ———————

5.4.5 Water Column Phosphorus Concentrations

Two of the three relationships just described for predicting algal biomass are predicated on phosphorus levels within the impoundment. Often, phosphorus data will be available only in inflowing water, and a mechanism is therefore needed to estimate impoundment phosphorus levels from phosphorus loading. Additionally, it may be desired to evaluate the effect of changes in loading on impoundment phosphorus levels.

Lorenzen, et al. (1976) developed a phosphorus budget model (Figure V-27) which may be used to estimate water column and sediment bound phosphorus in a fully mixed system. A mass balance on both sediment and water column phosphorus concentrations yields the coupled differential equations:

$$\frac{dC_w}{dt} = \frac{M}{V} + \frac{K_2AC_s}{V} - \frac{K_1AC_w}{V} - \frac{C_wQ}{V} \qquad \text{(V-17)}$$

$$\frac{dC_s}{dt} = \frac{K_1AC_w}{V_s} - \frac{K_2AC_s}{V_s} - \frac{K_1K_3AC_w}{V_s} \qquad \text{(V-18)}$$

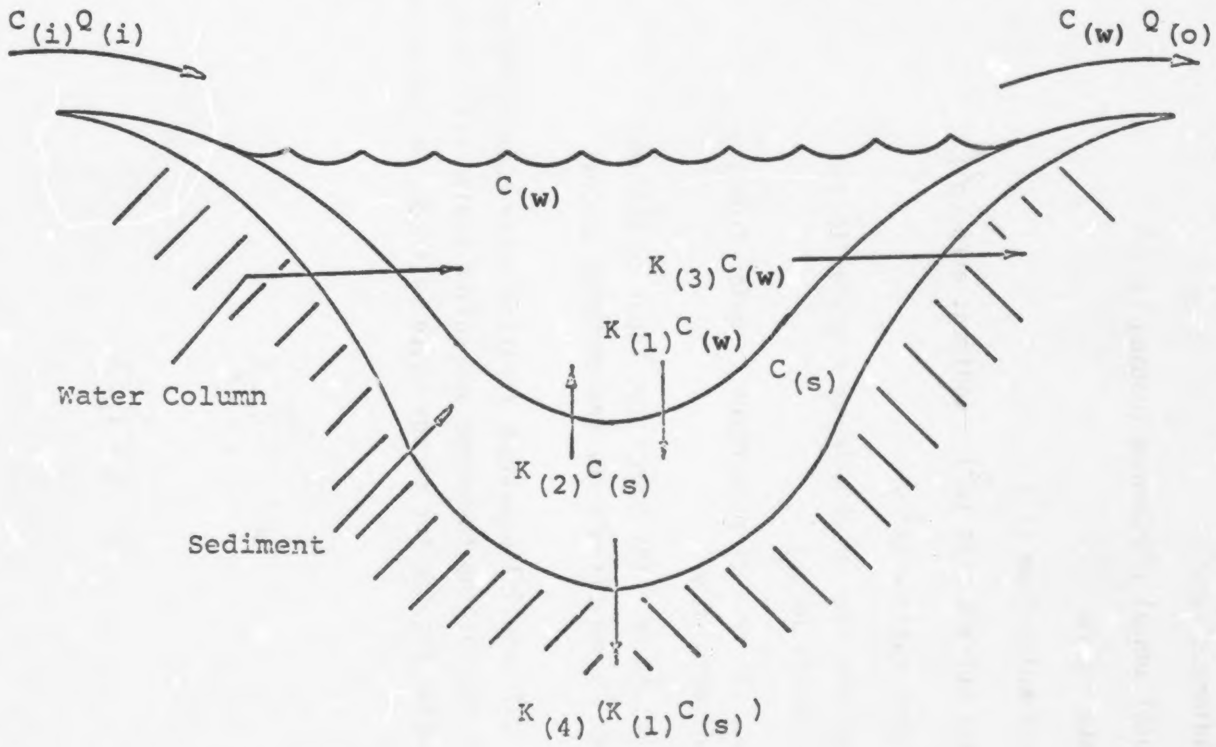

$C_{(i)}Q_{(i)}$
$C_{(w)}\ Q_{(o)}$
$C_{(w)}$
$K_{(3)}C_{(w)}$
$K_{(1)}C_{(w)}$
$C_{(s)}$
Water Column
$K_{(2)}C_{(s)}$
Sediment
$K_{(4)}(K_{(1)}C_{(s)})$

FIGURE V-27 CONCEPTUALIZATION OF PHOSPHORUS BUDGET MODELING (LORENZEN ET AL., 1976)

C_w = average annual total phosphorus concentration in water column (g/m^3)

C_s = total exchangeable phosphorus concentration in the sediments (g/m^3)

M = total annual phosphorus loading (g/yr)

V = lake volume (m^3)

V_s = sediment volume (m^3)

A = lake surface area (m^2) ≃ sediment area (m^2)

Q = annual outflow (m^3/yr)

K_1 = specific rate of phosphorus transfer to the sediments (m/yr)

K_2 = specific rate of phosphorus transfer from the sediments (m/yr)

K_3 = fraction of total phosphorus input to sediment that is unavailable for the exchange process

When the differential equations relating water column phosphorus to the various controlling phenomena are solved analytically, the following equation results for steady-state water column phosphorus concentration:

$$C_w = \frac{C_{in}}{1 + \frac{K_1 K_3 A}{Q}} \qquad \text{(V-19)}$$

or

$$C_w = \frac{M}{Q + K_1 K_3 A} \qquad \text{(V-20)}$$

where

C_w = steady-state water column phosphorus concentration in ppm

C_{in} = steady-state influent phosphorus concentration in ppm

The steady-state sediment phosphorus concentration is then given by:

$$C_s = \frac{C_{in}K_1(1 - K_3)}{K_2(1 + (K_1 K_3 A/Q))} \qquad \text{(V-21)}$$

It is important to observe that these relationships are valid only for steady-state conditions. Where phosphorus loading is changing with time, where sediment deposition or physical characteristics are changing, or where there are long-term changes in physical conditions, the steady-state solutions are not applicable.

Lorenzen applied the model to Lake Washington data and obtained very good results. With their data set, the most satisfactory coefficients had the following values:

K_1 = 43 m/yr

K_2 = 0.0014 m/yr

K_3 = 0.5

It should be recognized, however, that this model is relatively untested and that coefficient values for other impoundments will vary from those cited here.

Based upon the conceptualization (see Figure V-27), it is reasonable that the coefficients interact. For example, K_1, the rate of phosphorus uptake by the sediment must be related to the rate of phosphorus release by the sediment. The model requires however, that the product K_1K_3 be constant. The value used by Lorenzen, et al. was 21.6. As they point out, the coefficients must satisfy certain conditions, specifically those established by the derived equations. The equations are:

$$C_w = \frac{M}{Q + K_1K_3A} \tag{V-22}$$

and

$$\frac{C_w}{C_s} = \frac{K_2}{K_1(1-K_3)} \tag{V-23}$$

From (V-22)

$$K_1K_3 = \frac{M-QC_w}{C_wA} \tag{V-24}$$

Computation of K_1, therefore, requires a value for K_3. This coefficient, (K_3) unfortunately, is usually unavailable. It represents the fraction of phosphorus entering the sediment which is not available to return to the water column. Phenomena contributing to this are steady-state sediment accumulation, and steady-state chemical precipitation of phosphorus, such as with iron to form $Fe_3(PO_4)_2 \cdot 8H_2O$ (vivianite). Lorenzen's value for Lake Washington was 50%. Because the fraction is likely to vary significantly from system to system and because the coefficient is difficult to evaluate, the planner is advised to use 30% as the lower limit, 50% as a probable value, and 70% as an upper limit for estimating sediment phosphorus content. The water column concentration is independent of changes in K_3 because the product of K_1 and K_3 is a constant.

Using Equation (V-24), K_3 uniquely defines K_1. Then, from Equation (V-23),

$$K_2 = \frac{C_w K_1 (1-K_3)}{C_s}$$

K_2 is therefore also defined by fixing K_3, providing C_w and C_s are known.

EXAMPLE V-14

A Comprehensive Example
Impoundment Water Column Phosphorus

What will be the steady-state concentration of phosphorus in the water column of Lake Jones following diversion of flow? How will this affect algal abundance? A first step might be to use the Lorenzen model to evaluate K_1 and K_3 and determine whether the impoundment is at steady state with respect to phosphorus levels in the water column and sediment. Generally, this is the case where K_1K_3 lies in the range of 20 to 40. If K_1K_3 is outside of this range, field data should be obtained for current water column phosphorus.

Available Data (prior to diversion):

Inflows:

	Mean Annual Flow, cfs	Mean P, mg/l
1. Janes River	75	.15
2. Jennies River	22	.07
3. Johns Creek	5	.21
4. Direct stormwater influx	(nominal, may be disregarded)	

Lake Jones:

Area, A	20 miles^2 = 5.6×10^8ft^2 =5.2×10^7m^2
Volume, V.	3.08×10^{11}ft^3 = 8.73×10^9m^3
Sediment volume, V_s	Irrelevant for steady-state solution
Phosphorus (water column)	.01 mg/l

$$K_3 = 0.5$$

$$K_1 = \frac{M-QC_w}{K_3C_wA}$$

$$M = \left[\left(\frac{75\text{ft}^3}{\text{Sec}} \times \frac{0.15\text{ mg}}{\ell}\right) + \left(\frac{22\text{ft}^3}{\text{Sec}} \times \frac{.07\text{ mg}}{\ell}\right) + \left(\frac{5\text{ft}^3}{\text{Sec}} \times \frac{.21\text{ mg}}{\ell}\right)\right]$$

$$\times \frac{28.31\ell}{\text{ft}^3} \times \frac{1\text{ g}}{1000\text{ mg}} \times \frac{3.16\times10^7\text{sec}}{\text{yr}}$$

$$M = 1.24\times10^7 \text{ gP yr}^{-1}$$

$$Q = \frac{(75+22+5)\text{ft}^3}{\text{sec}} \times \frac{3.16\times10^7\text{sec}}{\text{yr}} = \frac{3.22\times10^9\text{ft}^3}{\text{yr}} = \frac{9.13\times10^7\text{m}^3}{\text{yr}}$$

$$C_w = .01\text{ mg/l} = .01\text{ g/m}^3$$

$$K_1 = \left(\frac{1.24\times10^7\text{gP}}{\text{yr}} - \frac{9.13\times10^7\text{m}^3}{\text{yr}} \times \frac{0.01\text{g}}{\text{m}^3}\right)\Big/$$

$$\left(.5 \times \frac{.01\text{g}}{\text{m}^3} \times 5.2 \times 10^7\text{m}^2\right) = 44\frac{\text{m}}{\text{yr}}$$

$$K_1K_3 = 44 \times 0.5 = 22$$

This approach, therefore, does not give reason to suspect non steady-state conditions for water column phosphorus. Assuming there are no other reasons for suspecting non steady-state phosphorus levels, it is feasible to estimate the new water column P concentration after diversion.

The diversion, which is for irrigation purposes, has resulted in lowering the mean annual inflow from Jennies River to 1 cfs with an average annual phosphorus concentration of 0.01 mg/l. Additionally, there is a reduction of flow in Janes River to 55 cfs. The new phosphorus concentration in the water column can be computed as follows:

$$Q = 55+22+1 = 78\text{cfs} \times \frac{3.16\text{x}10^7\text{sec}}{\text{yr}} = 2.46\text{x}10^9\ \frac{\text{ft}^3}{\text{yr}} = 6.98\text{x}10^7\ \frac{\text{m}^3}{\text{yr}}$$

Compute the net equivalent inflow as M:

$$M_{new} = \left[\left(\frac{55\text{ft}^3}{\text{sec}} \times \frac{.15\text{mg}}{1} + \frac{1\text{ft}^3}{\text{Sec}} \times \frac{.01\ \text{mg}}{1} + \frac{5\ \text{ft}^3}{\text{Sec}} \times \frac{.21\ \text{mg}}{1}\right)\right]$$

$$\times\ \frac{28.31\ \text{l}}{\text{ft}^3} \times \frac{1\ \text{g}}{1000\ \text{mg}} \times \frac{3.16\text{x}10^7\text{sec}}{\text{yr}} = 8.33\text{x}10^6\text{gPyr}^{-1}$$

Then, applying Equation V-22,

$$C_{w(new)} = \frac{8.33\text{x}10^6}{6.98\text{x}10^7+22\text{x}5.2\text{x}10^7} = 6.9\text{x}10^{-3}\ \text{mgl}^{-1}$$

Compare the new water column concentration, .0069, to the old value, .01 mg/l. Now refer to the methods presented earlier in this section. To apply the Vollenweider relationship, first to the pre-diversion status of Lake Jones, compute q_s:

$$q_s = \frac{\bar{Z}}{\tau_w}$$

$$\bar{Z} = \frac{8.73\text{x}10^9\text{m}^3}{5.2\text{x}10^7\text{m}^2} = 168\ \text{m}$$

$$\tau_w = 8.73\text{x}10^9 m^3/9.13\text{x}10^7 m^3 yr^{-1} = 95.6 \text{ yr}$$

$$q_s = 168/95.6 = 1.76 \text{ m yr}^{-1}$$

Compute phosphorus loading:

$$L_p = \frac{M}{A}$$

$$L_p = \frac{1.24\text{x}10^7 \text{ g yr}^{-1}}{5.2\text{x}10^7 \text{ m}^2} = 0.24 \text{ gm}^{-2}\text{yr}^{-1}$$

Referring to Figure V-23 with $q_s = 1.76$ and $L_p = 0.24$, one can see that this lake may have eutrophication problems under pre-diversion conditions.

After the diversion,

$$\tau_w = \frac{8.73\text{x}10^9 m^3}{6.98\text{x}10^7 m^3/yr} = 125 \text{ yr}$$

Assuming the lake depth is not materially changed over the short term,

$$q_s = 168/125 = 1.34 \frac{m}{yr}$$

For the new conditions,

$$M = 8.33 \text{ x } 10^6 \text{ gP yr}^{-1}$$

$$L_p = \frac{8.33\text{x}10^6 \text{ g yr}^{-1}}{5.2\text{x}10^7 \text{ m}^2} = .16 \text{ gP/m}^2\text{yr}$$

Now, according to the Vollenweider plot (Figure V-23), this is in the region between "dangerous" and "permissible" - the mesotrophic region. Under the new circumstances, algal blooms are less likely than before the flow diversions were established, but blooms are by no means to be ruled out.

Turning now to an estimate of algal biomass under pre-diversion conditions,

$$\text{chl } \underline{a} = 0.08(P)^{1.5}$$

$$\text{chl } \underline{a} = 0.08\ (0.01\times1000)^{1.5} = 2.5\ \text{mg/m}^3$$

$$\text{Dry algal biomass} \simeq 2.5\text{x}33 = 82.5\ \text{mg/m}^3$$

Under post-diversion conditions,

$$\text{chl } \underline{a} = 0.08(.0069\text{x}1000)^{1.5} = 1.5\ \text{mg/m}^3$$

$$\text{Dry algal biomass} \simeq 1.5\text{x}33=49.5\ \text{mg/m}^3$$

Note that these low levels of chlorophyll a almost certainly mean that the lake is oligotrophic, and that the Vollenweider method suggests worse conditions than may actually exist in this case.

——————————— END OF EXAMPLE V-14 ———————————

5.5 IMPOUNDMENT DISSOLVED OXYGEN

Organic substances introduced into an impoundment with inflowing water, falling onto its surface, or generated in the water column itself, may be oxidized by indigenous biota. The process consumes oxygen which may, in turn, be replenished through surface reaeration, photosynthetic activity, or dissolved oxygen in inflowing water. The dynamic balance between DO consumption and replenishment determines the net DO concentration at any point in time and at any location within the water column.

BOD exertion is not the only sink for DO. Table V-9 lists important sources and sinks of impoundment dissolved oxygen.

TABLE V-9

SOURCES AND SINKS OF
IMPOUNDMENT DISSOLVED OXYGEN

Sources	Sinks
Photosynthesis	Water Column BOD
Atmospheric reaeration	Benthic BOD
Inflowing water	Chemical oxidation
Rainwater	Deoxygenation at surface
	Plant and animal respiration

Many of the processes listed in Table V-9 have a complex nature. For example, the atmospheric reaeration rate is dependent in part upon the near-surface velocity gradient over depth. The gradient, in turn, is influenced by the magnitude, direction, and duration of wind, as well as the depth and geometry of the impoundment.

Photosynthetic rates are affected by climatological conditions, types of cells photosynthesizing, temperature, and a number of biochemical and biological factors. Exertion of BOD is dependent upon the kind of substrate, temperature, dissolved oxygen concentration, presence of toxicants, and dosing rate.

Despite this degree of complexity, a number of excellent models of varying degrees of sophistication have been developed which include simulation of impoundment dissolved oxygen.

5.5.1 Simulating Impoundment Dissolved Oxygen

Because an unstratified impoundment generally may be considered as a slow-moving stream reach, only stratified impoundments are of interest here. For estimating DO in unstratified impoundments, one should refer to the methods described in Chapter 4.

To understand the phenomena affecting dissolved oxygen in a stratified impoundment and to gain an appreciation of both the utility and limitations of the approach presented later, it is useful to briefly examine a typical dissolved oxygen model. Figure V-28 shows a geometric representation of a stratified impoundment. As indicated in the diagram, the model segments the impoundment into horizontal layers. Each horizontal layer is considered fully mixed at any point in time, and the model advects and diffuses mass vertically into and out of each layer. The constituents and interrelationships modeled are shown schematically in Figure V-29.

The phenomena usually taken into account in an impoundment DO model include:

- Vertical advection
- Vertical diffusion
- Correction for element volume change
- Surface replenishment (reaeration)
- BOD exertion utilizing oxygen
- Oxidation of ammonia
- Oxidation of nitrite
- Oxidation of detritus
- Zooplankton respiration
- Algal growth (photosynthesis) and respiration
- DO contribution from inflowing water
- DO removal due to withdrawals

Many of the processes are complex and calculations in detailed models involve simultaneous solution of many cumbersome equations.

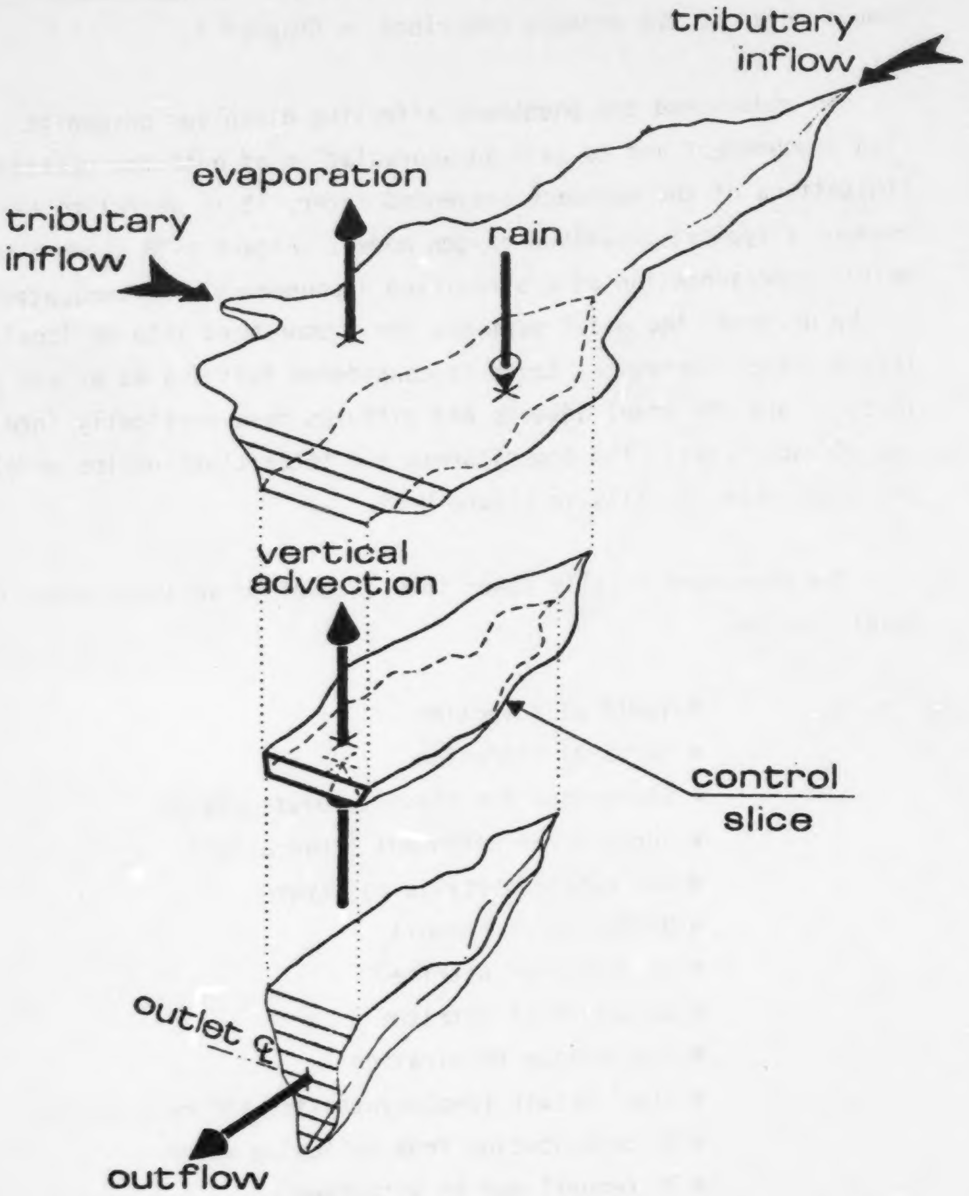

tributary
inflow
evaporation
tributary
inflow
rain
vertical
advection
control
slice
outlet ℄
outflow

Figure V-28 Geometric Representation of a Stratified Impoundment (From Hec, 1974)

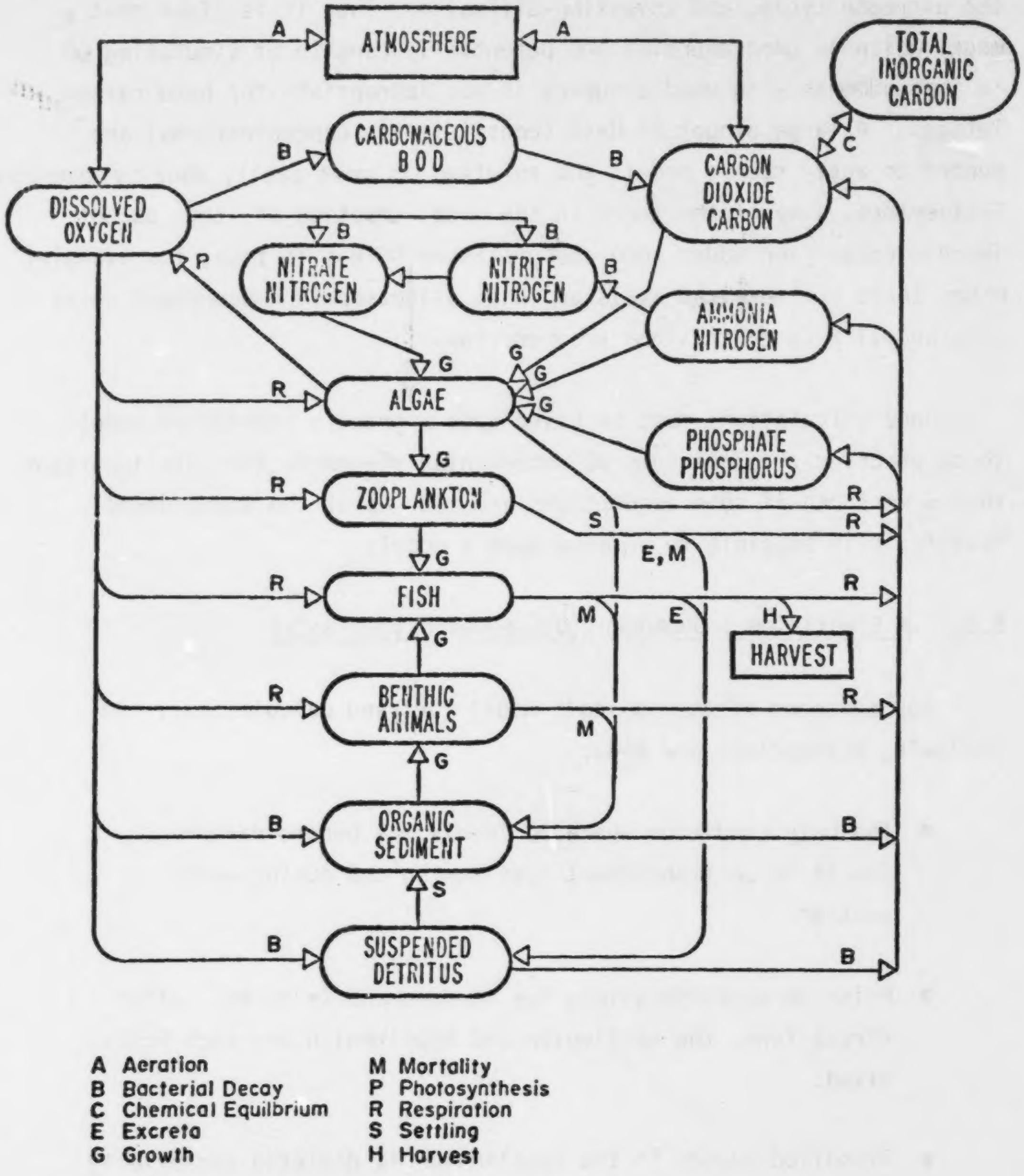

ATMOSPHERE
TOTAL INORGANIC CARBON
CARBONACEOUS BOD
CARBON DIOXIDE CARBON
DISSOLVED OXYGEN
NITRATE NITROGEN
NITRITE NITROGEN
AMMONIA NITROGEN
ALGAE
PHOSPHATE PHOSPHORUS
ZOOPLANKTON
FISH
HARVEST
BENTHIC ANIMALS
ORGANIC SEDIMENT
SUSPENDED DETRITUS
A
B
C
E
G
H
M
P
R
S
E,M
A Aeration
B Bacterial Decay
C Chemical Equilbrium
E Excreta
G Growth
M Mortality
P Photosynthesis
R Respiration
S Settling
H Harvest

FIGURE V-29 QUALITY AND ECOLOGIC RELATIONSHIPS (FROM HEC, 1974)

Among the processes simulated are zooplankton-phytoplankton interactions, the nitrogen cycle, and advection-diffusion. Thus it is clear that a model which is comprehensive and potentially capable of simulating DO in impoundments with good accuracy is not appropriate for hand calculations. A large amount of data (coefficients, concentrations) are needed to apply such a model, and solution is most easily done by computer. Furthermore, some of the terms in the model equation of state do not improve prediction under some circumstances. This is true, for example, where there are no withdrawals or in an oligotrophic impoundment where chlorophyll _a_ concentrations are very low.

Hand calculations must be based upon a greatly simplified model to be practical. Since some DO-determining phenomena are more important than others and if some assumptions are made about the impoundment itself, it is feasible to develop such a model.

5.5.2 A Simplified Impoundment Dissolved Oxygen Model

For purposes of developing a model for hand calculations, the following assumptions are made:

- The only condition where DO levels may become dangerously low is in an impoundment hypolimnion and during warm weather.

- Prior to stratification, the impoundment is mixed. After strata form, the epilimnion and hypolimnion are each fully mixed.

- Dissolved oxygen in the hypolimnion is depleted essentially through BOD exertion. Significant BOD sources and sinks to the water column prior to stratification are algal mortality, BOD settling, and outflows. A minor source is influent BOD. Following formation of strata, sources and sinks of BOD are BOD settling out onto the bottom, water column BOD at the time of stratification, and benthic BOD.

- Photosynthesis is unimportant in the hypolimnion as a source of DO.

- Once stratification occurs (a thermocline gradient of 1°C per meter of depth or greater) no mixing of thermocline and hypolimnion waters occurs.

- BOD loading to the unstratified impoundment and to the hypolimnion are in steady-state for the computation period.

5.5.2.1 Estimating a Steady-State BOD Load to the Impoundment

Equation V-25 is an expression to describe the rate of change of BOD concentration as a function of time:

$$\frac{dC}{dt} = k_a - k_s C - k_1 C - \frac{QC}{V} \qquad \text{(V-25)}$$

where

C = the concentration of BOD in the water column in $\mathrm{mgl^{-1}day^{-1}}$

k_a = the mean rate of BOD loading from all sources in $\mathrm{mgl^{-1}}$

k_s = the mean rate of BOD settling out onto the impoundment bottom in $\mathrm{day^{-1}}$

k_1 = the mean rate of decay of water column BOD in $\mathrm{day^{-1}}$

Q = mean export flow rate in $\mathrm{lday^{-1}}$

V = impoundment volume in liters

Integrating Equation V-25 gives:

$$C_t = \frac{(k_a + k_b C_o)e^{(k_b t)} - k_a}{k_b} \qquad \text{(V-26)}$$

where

C_t = concentration of BOD at time t
C_o = initial concentration of BOD
$k_b = -k_s - k_1 - \frac{Q}{V}$

To estimate the steady-state loading of BOD, we set dc/dt = 0 and obtain

$$C_{ss} = -\frac{k_a}{k_b} \qquad \text{(V-27)}$$

where

C_{ss} = steady-state water column BOD

Thus Equation (V-27) may be used to estimate a steady-state water column BOD concentration and Equation (V-26) may be used to compute BOD as a function of time, initial concentration of BOD, and the various rates.

5.5.2.2 Rates of Carbonaceous and Nitrogenous Demands

The rate of exertion of BOD and therefore the value of k_1 is dependent upon a number of physical, chemical, and biological factors. Among these are temperature, numbers and kinds of microorganisms, dissolved oxygen concentration, and the kind of organic substance involved. Nearly all of the biochemical oxygen demand in impoundments is related to decaying plant and animal matter. All such material consists essentially of carbohydrates, fats, and proteins along with a vast number of minor constituents. Some of these are rapidly utilized by bacteria, for example, the simple sugars, while some, such as the celluloses, are metabolized slowly.

Much of the decaying matter in impoundments is carbonaceous.

Carbohydrates (celluloses, sugars, starches) and fats are essentially devoid of nitrogen. Proteins, on the other hand, are high in nitrogen (weight of carbon/weight of nitrogen ≃ 6) and proteins therefore represent both carbonaceous and nitrogenous demands.

The rate of exertion of carbonaceous and nitrogenous demands differ. Figure V-30, which shows the difference graphically and as a function of time and temperature, may be considered to represent the system response to a slug dose of mixed carbonaceous and nitrogenous demands. In each two-section curve, especially where concentrated carbonaceous wastes are present, the carbonaceous demand is exerted first, and this represents the first stage of deoxygenation. Then nitrifiers increase in numbers and ammonia is oxidized through nitrite and ultimately to nitrate. This later phase is called the second phase of deoxygenation.

BOD decay (either nitrogenous or carbonaceous alone) may be represented by first order kinetics. That is, the rate of oxidation is directly proportional to the amount of material remaining at time t.

$$\frac{dC}{dt} = -kC \qquad \text{(V-28)}$$

The rate constant, k, is a function of temperature, bacterial types and numbers, composition and structure of the substrate, presence of nutrients and toxicants, and a number of other factors. The value of the first stage constant k_1 was first determined by Phelps in 1909 for sewage filter samples. The value was 0.1 (Camp, 1968). More recent data show that at 20°C, the value can range from 0.01 for slowly metabolized industrial waste organics to 0.3 for relatively fresh sewage (Camp, 1968).

The typical effect of temperature on organic reactions is to double reaction rates for each temperature rise of 15°C. The relationship for correcting k_1 for temperature is:

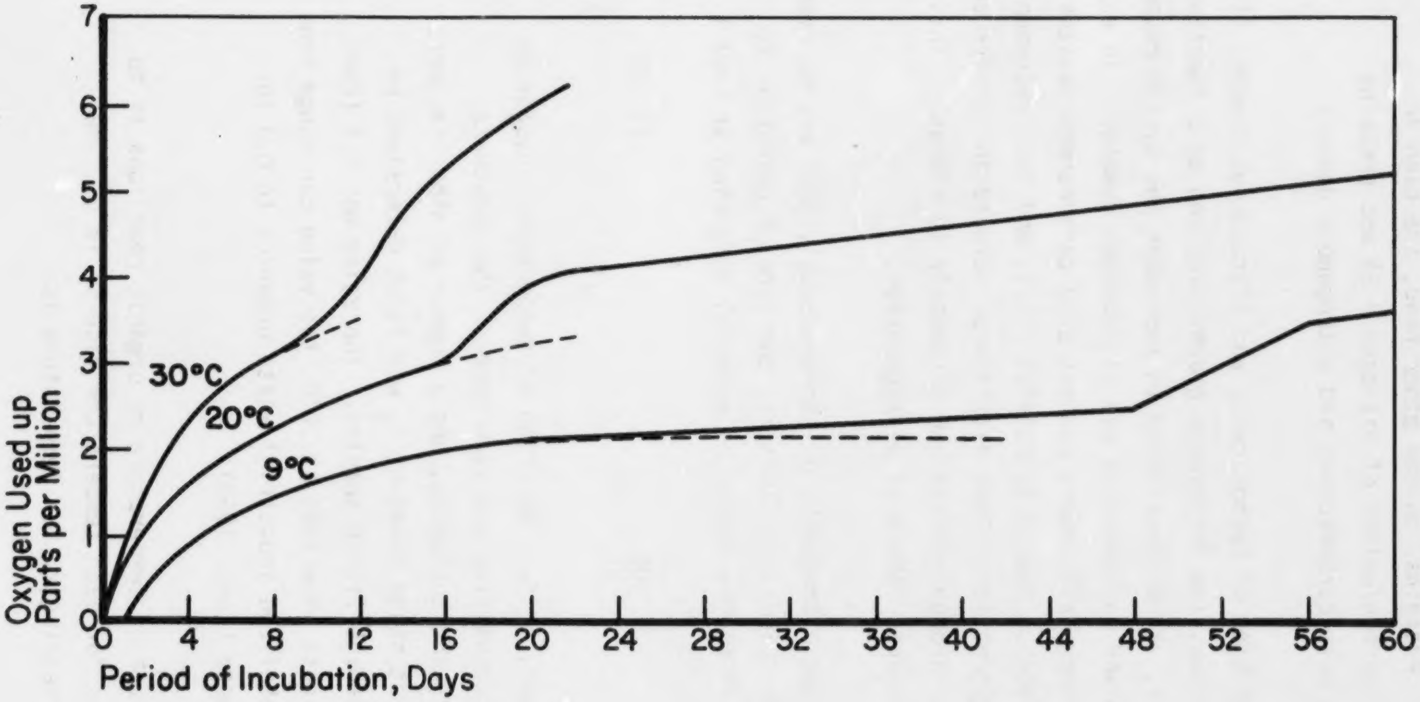


FIGURE V-30 RATE OF BOD EXERTION AT DIFFERENT TEMPERATURES SHOWING THE FIRST AND SECOND DEOXYGENATION STAGES

$$k_{1,(T)} = k_{1,(20°C)}\ \theta^{(T-20)} \qquad \text{(V-29)}.$$

where

T = the temperature of reaction

θ = correction constant = 1.047

However, Thereault has used a value for θ of 1.02, while Moore calculated values of 1.45 and 1.065 for two sewages and 1.026 for river water (Camp, 1968).

Streeter has determined the rate of the nitrification or second deoxygenation stage in polluted streams. At 20^{0}C, k_1 for nitrification is about 0.03 (Camp, 1968). Mobre found the value to be .06 at 20^{0}C and .035 at 10^{0}C (Camp, 1968). For purposes of this analysis, BOD exertion will be characterized as simple first order decay using a single rate constant.

Benthic demand, which is important in later computations, may vary over a wide range because in addition to the variability due to the chemical nature of the benthic matter, rates of oxidation are limited by upward diffusion rates of oxidizable substances through pores in the benthos. Since the nature of the sediment is highly variable, benthic oxygen demand rates very more than values for k_1 in the water column. In a study using sludges through which oxygenated water was passed, initial rates of demand ranged from 1.02 g/m^2 day (see Table V-10) for a sludge depth of 1.42 cm up to 4.65 g/m^2 day for a sludge depth of 10.2 cm (Camp, 1968). In that study, the values found were for initial demand since the sludge was not replenished. The rate per centimeter of sludge depth, then, can vary from a low of 0.46 g/m^2 day for 10.2 centimeter depth sludge up to 0.76 g/m^2 day for 1.42 centimeter depth sludge.

TABLE V-10

OXYGEN DEMAND OF BOTTOM DEPOSITS
(AFTER CAMP, 1968)

Benthic Depth (mean) cm	Initial Volume of Solids, kgm^{-2}	Initial Area Demand		$k_4(20^oC)$
		L (gm^{-2})	Initial Demand $gm^{-2}day^{-1}$	
10.2	3.77	739	4.65	.0027
4.75	1.38	426	3.09	.0031
2.55	0.513	227	1.70	.0032
1.42	0.188	142	1.08	.0033
1.42	0.188	134	1.02	.0033

The constant loading rate (k_a) used in Equation (V-25) is best estimated from historical data. Alternatively, inflow loading (see Chapter IV) and algal productivity estimates (this chapter) may be used. In the latter case, a value must be adopted for the proportion of algal biomass ultimately exerted as BOD. To a first approximation, k_a may be estimated using this value and adopting some percentage of maximal primary productivity (see Figure V-25). Thus,

$$k_a(\text{algae}) = SMP \times 10^{-3}/D \qquad \text{(V-30)}$$

where

k_a(algae) = algal contribution to BOD loading rate

S = stoichiometric conversion from algal biomass as carbon to BOD = 2.67

M = Proportion of algal biomass expressed as an oxygen demand (unitless)

P = Primary production in $mgCm^{-2}day^{-1}$

The difference between algal biomass and the parameter M representing the proportion of algal biomass exerted as BOD may be conceptualized as accounting for such phenomena as incorporation of algal biomass into fish tissue which either leaves the impoundment or is harvested, loss of carbon to the atmosphere as CH_4, and loss due to outflows.

The settling rate coefficient, k_s in Equation (V-25) must be estimated for the individual case. It represents the rate at which dead plant and animal matter (detritus) settles out of the water column prior to oxidation. Clearly, this coefficient is sensitive to the composition and physical characteristics of suspended matter and the turbulence of the system. Quiescence and large particle sizes in the organic fraction will tend to give high values for k_s while turbulence and small organic fraction particle sizes will give small values for k_s.

5.5.2.3 Estimating a Pre-Stratification Steady-State Dissolved Oxygen Level

Prior to stratification, the impoundment is assumed to be fully mixed. One of the important factors leading to this condition is wind stress, which also serves to reaerate the water. As a rule, unless an impoundment acts as a receiving body for large amounts of nutrients and/or organic loading, dissolved oxygen levels are likely to be near saturation during this period (D.J. Smith, pers. comm., November, 1976). Table V-11 shows saturation dissolved oxygen levels for fresh water as a function of temperature, and DO levels may be estimated accordingly.

TABLE V-11

SATURATION DO LEVELS IN FRESH WATER AS A
FUNCTION OF TEMPERATURE (CHURCHILL, ET AL., 1961)
(ATMOSPHERIC PRESSURE = 760 MM Hg, TDS = 0 mg/l)

Temperature	DO, ppm (Saturation)
0	14.65
5	12.79
10	11.27
15	10.03
20	9.02
25	8.18
30	7.44

5.5.2.4 Estimating Hypolimnion DO Levels

The final step in use of this model is preparation of a DO-versus-time plot for the hypolimnion (or at least estimation of DO at incipient overturn) and estimation of BOD and phosphorus loadings which result in acceptable hypolimnion DO levels. An equation to compute DO at any point in time during the period of stratification is

$$\frac{dO}{dt} = -k_1C-k_4L/D \qquad \text{(V-31)}$$

where

O = dissolved oxygen in ppm

k_4 = benthic decay rate in day^{-1}

L = areal BOD load in gm^{-2}

D = depth in m

The second term in the equation requires that an estimate be made of the magnitude of BOD loading in benthic deposits. To do this within the present framework, it is assumed that BOD settles out prior to stratification and that following stratification, surface velocities are adequately high and density gradients steep to ignore settling of mass into the hypolimnion. The pre-stratification benthic demand is a function of the amount of BOD settling and the rate of benthic BOD decay. Based upon the rate of settling used earlier in estimating a steady-state BOD concentration (Equation (V-25)) and rate of decay for conditions prior to stratification, the rate of benthic matter accumulation is:

$$\frac{dL}{dt} = k_s C_{ss} D - k_4 L \qquad \text{(V-32)}$$

where

C_{ss} = concentration of BOD in the water column in gm^{-3} at steady-state

The assumption of steady-state BOD concentration reduces Equation (V-32) to the same form as Equation (V-25) and integration gives:

$$L_t = \frac{(k_s D C_{ss} - k_4 L_0) e^{-k_4 t} - k_s D C_{ss}}{-k_4} \qquad \text{(V-33)}$$

For steady-state deposition ($dL/dt = 0$, $Dk_s C_{ss}$ = constant),

$$L_{ss} = \frac{k_s C_{ss} D}{k_4} \qquad \text{(V-34)}$$

where

L_{ss} = steady-state benthic BOD load in gm^{-2}

Application of Equation (V-34) with k_s and k_4 appropriately chosen for the month or two preceding stratification will give an estimate of the benthic BOD load upon stratification. Application of Equation (V-33) gives the response of L to different water column BOD (steady-state) loading rates and changes in rate coefficients.

After strata form, benthic matter decays while hypolimnion water column BOD decays and settles. The change in L over the period of stratification is

$$\frac{dL}{dt} = -k_4L + Dk_sC \tag{V-35}$$

Since

$$\frac{dC}{dt} = -k_sC - k_1C = -(k_1+k_s)C \tag{V-36}$$

and

$$C_t = C_o\, e^{-(k_1 + k_s)t} \tag{V-37}$$

$$\frac{dL}{dt} = -k_4L + Dk_sC_o\, e^{-(k_1 + k_s)t} \tag{V-38}$$

Then

$$L_t = \left(L_o + \frac{Dk_sC_o}{k_s+k_1-k_4}\right) e^{-k_4t} - \left(\frac{Dk_sC_o}{k_s+k_1-k_4}\right) e^{(-k_s-k_1)t} \tag{V-39}$$

Water column BOD in the hypolimnion is given by Equation (V-36) and the integrated form is Equation (V-37).

Note that k_s, the settling coefficient is equal to v_s/D where v_s is the settling velocity of the BOD, and D is the depth of the hypolimnion (or when the impoundment is unstratified, D is the depth of the entire impoundment).

The equation presented earlier (Equation V-31) for hypolimnion DO was:

$$\frac{dO}{dt} = -k_1C - k_4L/D$$

Equation (V-31) is not integrable in its present form, but since L and C are defined as functions of t (Equations (V-39) and (V-37) respectively), it is possible to determine dissolved oxygen in the water column. The equation for oxygen at time t is:

$$O_t = O_o - \Delta O_L - \Delta O_C \qquad \text{(V-40)}$$

where

Ot = dissolved oxygen at time t
O_o = dissolved oxygen at time t = 0
ΔO_L = dissolved oxygen decrease due to benthic demand
ΔO_C = dissolved oxygen decrease due to hypolinminion BOD

From Equation (V-39), and using L_{ss} as L_o and C_{ss} as C_o,

$$\Delta O_L = \left(\frac{L_{ss}}{D} + \frac{k_sC_{ss}}{k_s+k_1-k_4}\right)\left(1-e^{-k_4t}\right) - \left(\frac{k_sC_{ss}}{k_s+k_1-k_4}\right)\left(\frac{k_4}{k_s+k_1}\right)\left(1-e^{-(k_s+k_1)t}\right) \qquad \text{(V-41)}$$

and from Equation (V-37),

$$\Delta O_C = \frac{k_1C_{ss}}{k_1+k_s}\left(1-e^{-(k_1+k_s)t}\right) \qquad \text{(V-42)}$$

Solution of Equation (V-40) gives an estimated DO concentration in the hypolimnion as a function of time.

EXAMPLE V-15

Quiet Lake
(Comprehensive Example)

Quiet Lake is located a few miles south of Colton, New York. The lake is roughly circular in plan view (Figure V-31) and receives inflows from three tributaries. There is one natural outlet from the lake and one withdrawal used for quarrying purposes.

The first step in evaluation of lake hypolimnion DO levels is physical and water quality data collection. Table V-12 shows characteristics of Quiet Lake, Table V-13 shows tributary discharge data along with withdrawal and outflow levels, and Table V-14 provides precipitation and runoff information.

In order to evaluate hypolimnion DO as a function of time, the very first question to be answered is, does the impoundment stratify? If so, what are the beginning and ending dates of the stratified period, how deep is the upper surface of the hypolimnion, and what is its volume, and what is the distribution of hypolimnion mean temperatures during the period? To answer these questions, either use field observation data, or apply some computation technique such as that presented earlier in this section. Assuming that methods presented earlier are used, the selection of appropriate thermal profile curves hinges around three factors. These are

- Climate and location
- Hydraulic residence time, and
- Impoundment geometry

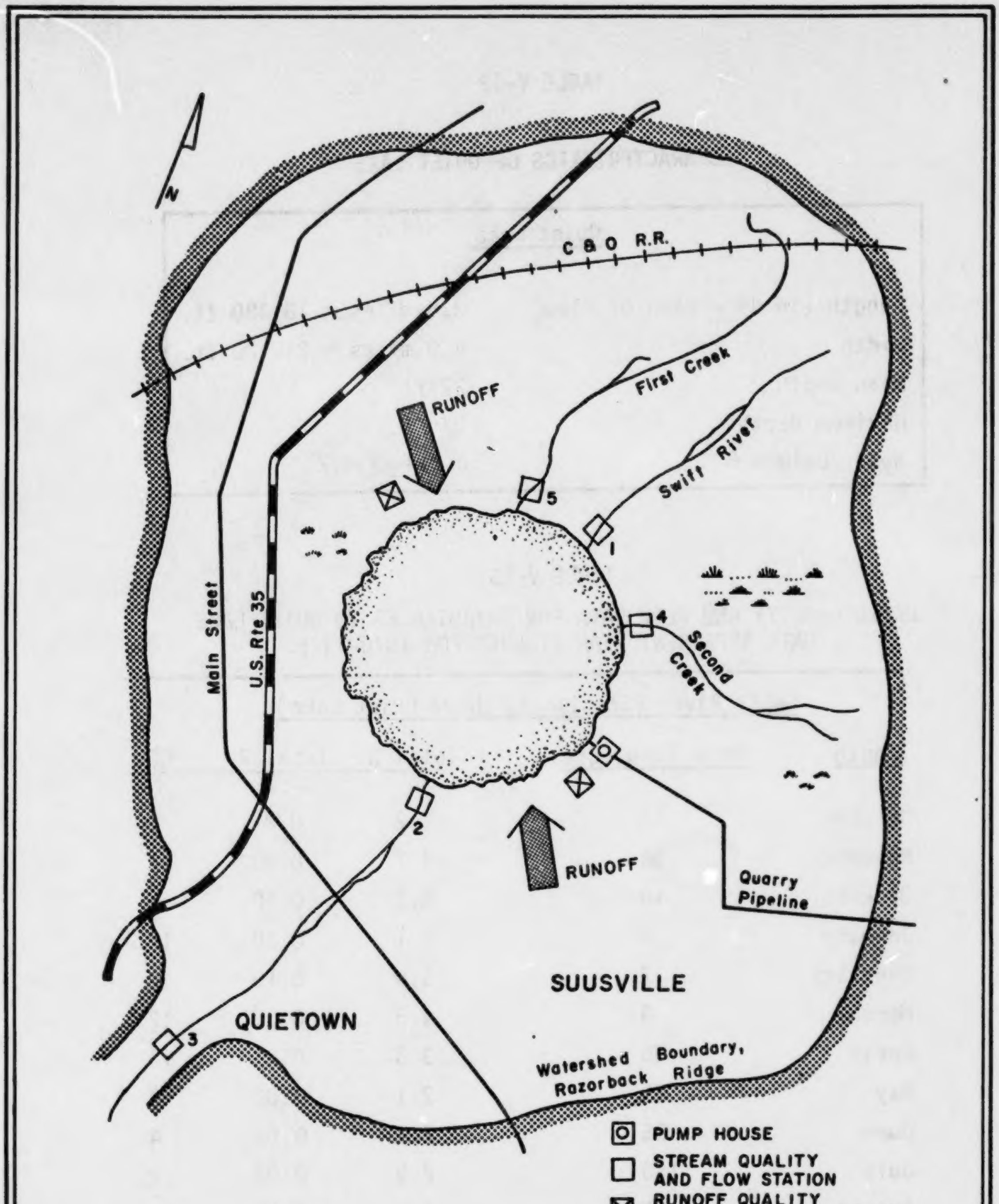

N
C & O R.R.
First Creek
Swift River
RUNOFF
5
1
4
Second Creek
Main Street
U.S. Rte 35
2
RUNOFF
Quarry Pipeline
SUUSVILLE
QUIETOWN
3
Watershed Boundary, Razorback Ridge
PUMP HOUSE
STREAM QUALITY AND FLOW STATION
RUNOFF QUALITY SAMPLING STATION
SAMPLES TAKEN FROM SMALL EROSION CHANNELS NEAR LAKE

FIGURE V-31. QUIET LAKE AND ENVIRONS

TABLE V-12

CHARACTERISTICS OF QUIET LAKE

Quiet Lake	
Length (in direction of flow)	3.5 miles = 18,480 ft.
Width	4.0 miles = 21,120 ft.
Mean Depth	22 ft.
Maximum Depth	27 ft.
Water Column P	$0.014 \leq P \leq .032$

TABLE V-13

WATER QUALITY AND FLOW DATA FOR TRIBUTARIES TO QUIET LAKE. DATA REPRESENT MEAN FIGURES FOR 1970-1975.

Swift River (Station 1, above Quiet Lake)

Month	Mean Flow, cfs	Total N	Total P	BOD
			ppm	
October	54	2.2	0.2	3
November	38	4.1	0.08	4
December	10	5.3	0.10	6
January	5	6.1	0.20	12
February	2	5.0	0.15	10
March	8	4.3	0.08	12
April	40	3.3	0.04	10
May	55	2.1	0.02	8
June	85	2.8	0.02	4
July	150	2.9	0.02	2
August	70	1.0	0.02	1
September	85	2.4	0.03	1

TABLE V-13 (Continued)

First Creek (Station 5)

Month	Mean Flow, cfs	Total N	Total P ppm	BOD
October	5	1.0	.01	0.5
November	3	2.0	.01	1.0
December	2	0.5	.02	1.5
January	2	1.2	.01	1.0
February	3	1.3	.02	0.8
March	4	2.3	.01	0.6
April	6	2.0	.01	0.5
May	8	1.8	.02	0.6
June	10	1.6	.01	0.8
July	8	1.4	.01	0.8
August	6	1.5	.00	1.0
September	4	0.8	.00	1.2

Second Creek (Station 4)

Month	Mean Flow, cfs	Total N	Total P ppm	BOD
October	14.0	15	.15	7
November	13.0	16	.08	8
December	12.5	10	.20	10
January	5.0	9	.15	7
February	1.2	12	.12	7
March	2.0	13	.10	6
April	2.5	8	.11	7
May	4.0	6	.07	9
June	8.0	5	.08	12
July	12.0	7	.20	3
August	8.0	6	.22	4
September	5.5	8	.25	8

TABLE V-13 (Continued)

Swift River (Stations 2 and 3) and Pumped Withdrawal

Month	Pumped Withdrawal, cfs	Mean Monthly Flow, cfs	
		Station 2	Station 3
October	22.6	69.5	77.0
November	22.0	50.0	55.0
December	3.5	20.0	22.0
January	1.2	7.5	9.0
February	0.8	1.2	1.4
March	0.4	9.1	10.1
April	12.0	44.5	48.75
May	24.0	63.2	69.5
June	30.7	100.0	110.0
July	89.5	168.5	184.8
August	29.8	80.6	88.5
September	43.9	91.3	100.25

Notes: All three tributaries have their headwaters within the shed. The net inflow-outflow to the groundwater is known to be close to zero in the two creeks. Swift River is usually about 10% effluent over its entire length (10% of flow comes into the river from the groundwater table).

TABLE V-14

PRECIPITATION AND RUNOFF DATA FOR QUIET LAKE WATERSHED. VALUES ARE MEANS OF DATA COLLECTED FROM BOTH STATIONS (SEE FIGURE V-31). THE WATERSHED HAS AN AREA OF 55 SQUARE MILES INCLUDING THAT OF THE LAKE

Month	Mean Total Monthly Precipitation, inches	Runoff Quality		
		Total N	Total P ppm	BOD
October	3.0	6.0	0.1	27
November	2.4	6.5	0.2	37
December	1.0	4.0	0.1	46
January	0.5	3.0	0.008	34
February	0.3	1.0	0.07	33
March	0.6	1.5	0.1	30
April	2.0	2.5	0.15	40
May	2.8	3.2	0.25	50
June	4.2	3.6	0.20	40
July	7.6	7.0	0.40	37
August	3.5	7.8	0.60	45
September	4.2	9.2	0.80	50
Total	32.1			

Note: Infiltration to the water table on a monthly basis accounts for roughly 30% of precipitation volume.

In terms of climate and location, the Quiet Lake area is similar to Burlington, Vermont. Examination of the Burlington plots from Appendix D reveals that a 20-foot maximum depth impoundment can stratify in an area shielded from the wind. The area surrounding Quiet Lake does provide good shielding, so the next task is to estimate the hydraulic residence time to select a specific set of plots.

Inspection of all Burlington plots indicates that stratification is likely to occur at most from May to August. Accordingly, for purposes of plot selection, we are most interested in a mean hydraulic residence time based on flows in the period from about March to August. Since hydraulic residence time (τ_w) is given by $\tau_w = V/Q$, we compute mean Q ($\bar{Q}$). $\bar{Q}$ represents the average of tributary inflows during this period, computed as follows:

$$\bar{Q} = \underset{\text{(Swift River)}}{\frac{8+40+55+85+150+70}{6}} + \underset{\text{(First Creek)}}{\frac{4+6+8+10+8+6}{6}} + \underset{\text{(Second Creek)}}{\frac{2+2.5+4+8+12+8}{6}}$$

$$\bar{Q} = 68+7+6.08 = 81.1 \text{ cfs}$$

However, in order to fully account for mass transport as well as properly estimate hydraulic residence time, one more factor should be considered. This is non-point inflow. At this point, we have enough information to estimate the stormwater contribution directly to Quiet Lake. In view of the available data, the computation is as follows:

$$Q_s = APK(1-L) - \left(\sum_{i=1}^{n} Q_i (1-I_i) \right)$$

where

Q_s = stormwater or non-point inflow in cfs (excluding rivers and creeks)

A = area of shed in square miles

n = number of tributaries

Q_i = monthly mean pickup (in cfs) in the ith tributary

P = monthly total precipitation, in inches per month

I_i = percent (expressed as a decimal) of flow contributed by exfiltration (from the water table into the channel)

L = the proportion of precipitation lost by infiltration into the soil (expressed as a decimal)

K = unit correction = 0.895 $ft^3 mo\ mi^{-2} in^{-1} sec^{-1}$

As an example, the computation for October is:

$$Q_s = 55\ mi^2 \times 3.0\ \frac{in}{mo} \times 0.896\ \frac{ft^3 mo}{mi^2 in\ sec} \times (1-0.3) -$$

$$\Big(54(1-0.1)+5(1-0.0)+14(1-0.0)+(77-69.5)(1-0.1)\Big) = 29.1\ cfs$$

Now, since we know the pumped withdrawal rates as well as the difference between flows at stations 2 and the sum of 1, 4, and 5, and that the impoundment surface is at steady-state over the mouth, we also can estimate the net infiltration rate from the lake into the groundwater. The infiltration rate is (again, for October):

$$\text{Net efflux} = Q_{(sta\ 1+4+5)} - Q_2 + Q_s - Q_w$$

$$= 73.0 - 69.5 + 29.1 - 22.6 = 10.0\ cfs$$

Note that the pickup in each channel above Quiet Lake is equal to the flow at the pertinent sampling station. This is the case because the three channels have their headwaters within the watershed. If one were concerned about a subshed with tributary headwaters above the subshed boundary, the difference in Q between each of stations 1, 4, and 5 and the respective flows at the upstream subshed boundary would be used.

To estimate hydraulic residence time add the mean stormwater contribution over the months of interest to that of the tributaries, as computed earlier. The individual stormwater computations are not shown. The method is as just described.

$$\bar{Q}_{total} = 81.1 + \frac{6.6+20.7+29.4+41.4+92.5+36.6}{6} = 119\ cfs$$

Then the hydraulic residence time is given by:

$$\tau_w = V/Q = \pi r^2 D/Q$$

$$r = \left[\frac{L+W}{4} \times 5280 \right]$$

where

L = length of the lake in mi.

W = width of the lake in mi.

D = mean depth in ft.

r = radius in ft.

$$\tau_w = 3.14 \times \left[\frac{3.5+4}{4} \times 5280 \right]^2 \times 22/119$$

$$= 5.69 \times 10^7 \text{ sec} = 658 \text{ days}$$

Accordingly, the infinite hydraulic residence time plots for a 20-foot deep, wind-protected, Burlington, Vermont, impoundment should suffice. Note that the entire impoundment volume was used in the above computation. Strictly, one should use the epilimnion volume during stratification. In this case, such a change would not alter selection of the plots because τ_w would still be greater than 200 days. A reproduction of the appropriate plot from Appendix D is presented in Figure V-32. As indicated, Quiet Lake is likely to be weakly stratified from May to August inclusive, with a thermocline temperature gradient of about 1°ft^{-1}. The hypolimnion should extend downward to the bottom from a depth of about 3-1/2 meters, giving a mean hypolimnion depth of

$$D_H = \frac{22 \text{ ft}}{3.28 \text{ ft m}^{-1}} - 3.5 \text{ m} = 3.2 \text{ meters}$$

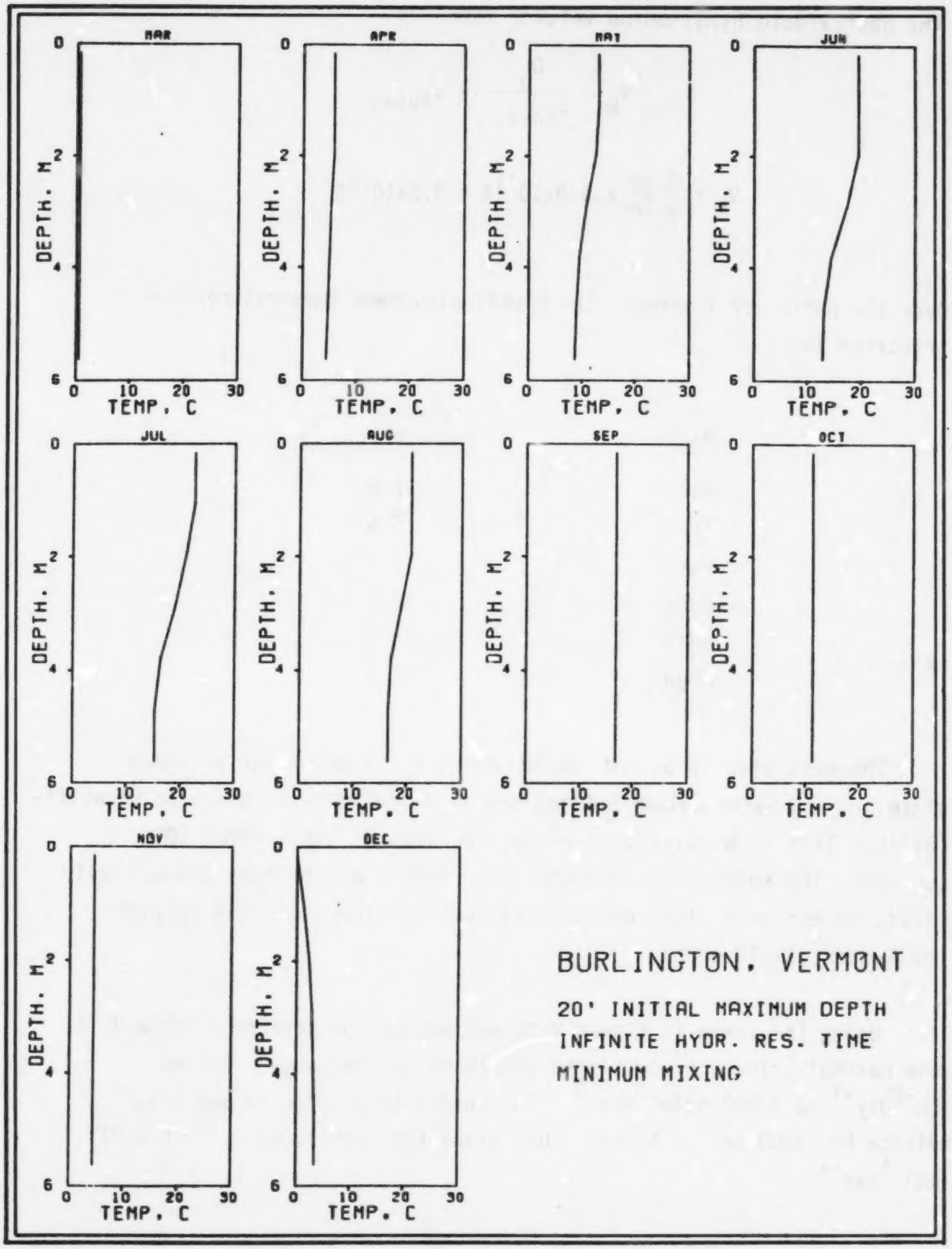


FIGURE V-32 THERMAL PROFILE PLOTS FOR USE IN QUIET LAKE EXAMPLE

The approximate hypolimnion volume, then, is

$$V_H = \frac{D_H}{D_{Total}} \times V_{Total}$$

$$V_H = \frac{3.2m}{6.7m} \times 1.9x10^{11}\ell = 9.2x10^{10}\ell$$

Over the period of interest, the hypolimnion mean temperature distribution is:

Month	Mean Temperature, °C
March	2.0
April	5.5
May	9.5
June	12.5
July	14.0
August	15.5

The next step in use of the DO model is to determine a steady-state or mean water column BOD loading (k_a) and DO level prior to stratification. This is a multi-step process because of the several BOD sources. The sources are tributaries, runoff, and primary productivity. First, we estimate algal productivity using methods of this chapter (or better, field data).

Using the curve in Figure V-26 and phosphorus data from Table V-12, the maximal primary productivity should be in the range 1,400 mg $Cm^{-2}day^{-1}$ to 1,900 mg$Cm^{-2}day^{-1}$. To convert to loading in $mgl^{-1}day^{-1}$, divide by ($1000\ lm^{-3} \times 6.7m$). This gives the loading as 0.21 to 0.28 $mgl^{-1}day^{-1}$.

Now assuming that maximal productivity occurs at about 30^{o}C and that productivity rates obey the same temperature rule as BOD decay, temperature-adjusted estimates of productivity rates can be made. Using the maximal rate range of 0.21 to 0.28 $mgl^{-1}day^{-1}$, the adjusted rates are:

$$\begin{aligned} \text{Productivity} &= (0.21,\ 0.28) \times 1.047^{(3.75-30)} \\ &= (.06,.08)\ mgl^{-1}\ day^{-1} \end{aligned}$$

Then, according to Equation (V-30) and assuming M = 1, k_a due to algae is estimated by:

$$k_a(\text{algae}) = 2.67 \times (.06,\ .08) = (.16,\ .21)\ mgl^{-1}day^{-1}$$

The next contributor to water column BOD is BOD loading of inflowing waters. The value to be computed is the loading in milligrams per liter of impoundment water per day.

$$\text{Daily BOD loading rate} = \left(\sum_{i=1}^{n} \sum_{j=1}^{L_i} d_i Q_{i,j}\ C_{i,j} \right) \Big/ V\sum_{k=1}^{n} d_k$$

where n = the number of time periods of measurement
V = volume of impoundment in liters
d = the number of days per time period
L = the number of inflows

For all inflows, the value is therefore approximately:

$$k_{a(\text{Trib})} = (\underset{\text{(Swift River)}}{2185} + \underset{\text{(First Creek)}}{48.3} + \underset{\text{(Second Creek)}}{643.9} + \underset{\text{(Storm water Runoff)}}{14240}) \times \underset{\text{(Units Conversion)}}{2.45\text{x}10^6} \times \underset{\text{(Impoundment Volume)}}{\frac{1}{1.9\text{x}10^{11}}} = 0.22\ mgl^{-1}day^{-1}$$

Now, summing the two contributions:

$$k_a = k_{a(\text{algae})} + k_{a(\text{Trib})}$$
$$k_a = (.16,\ .21) + .22 = (.38,\ .43)\ mgl^{-1}day^{-1}$$

The value of k_1 will be assumed as 0.1 at 20^oC with θ in Equation (V-29) equal to 1.047. Then at 3.75^oC,

$$k_{1(3.75^oC)} = k_{1(20^oC)} \times 1.047^{(3.75-20)}$$

$$= .1 \times 1.047^{(-16.25)} = 0.047$$

Now $Q_{(discharge)}$ (mean for March and April) and V are known, with

$$Q_{(discharge)} = 26.8 \text{ (Swift River, Station 2)}$$

$$+ 6.2 \text{ (pumped withdrawal)} \times \frac{28.32\ell}{ft^3} = 935\ell \ sec^{-1}$$

$$V = 1.9 \times 10^{11}\ell$$

Then $$C_{ss} = \frac{.38, .43}{(.03+.047+(935/1.9\text{x}10^{11}))} = 4.94, 5.58$$

For further computations, $C_{ss} = 5.25$ will be assumed.

Since k_s has been defined as .03, a steady-state areal concentration of benthic BOD prior to stratification can be estimated. If $k_{4(20^oC)} = .003$ and $C_{ss} = 5.25$, using Equation (V-34),

$$L_{ss} = \frac{k_s C_{ss} D}{k_{4(3.75^oC)}}$$

$$K_{4(3.75^oC)} = .003\text{x}1.047^{(3.75-20)} = .0014$$

$$L_{ss} = \frac{.03\text{x}5.25\text{x}6.7}{.0014} = 754 \ gm^{-2}$$

The next step in evaluating hypolimnion DO depression is to estimate pre-stratification DO levels. If we assume saturation at the mean temperature in April (5.5^oC), the dissolved oxygen concentration at onset of strata should be about 12.7 (from Table V-11).

Now we have all values needed to plot hypolimnion DO versus time using Equations (V-40) through (V-42).

Using $L_o = L_{ss}$

$C_o = C_{ss}$

$k_1 = 0.1 \times 1.047^{(9.5-20)} = .062$, ($T = 9.5^{\circ}C$ for May)

$k_s = 0.03$,

$k_4 = .003 \times 1.047^{(9.5-20)} = .002$, and

$t = 5$ days,

and applying Equation (V-42),

$$\Delta O_c = \frac{k_1 C_{ss}}{k_1 + k_s} \left(1 - e^{-(k_1 + k_s)t} \right)$$

$$\Delta O_c = \frac{0.062 \times 5.25}{0.062 \times 0.03} \left(1 - e^{-(0.062+0.03)5} \right) = 1.30$$

Then, according to Equation (V-41),

$$\Delta O_L = \left(\frac{L_{ss}}{D} + \frac{k_s C_{ss}}{k_s + k_1 - k_4} \right) \left(1 - e^{-k_4 t} \right) - \left(\frac{k_s C_{ss}}{k_s + k_1 - k_4} \right) \left(\frac{k_4}{k_s + k_1} \right) \left(1 - e^{-(k_s + k_1)t} \right)$$

$$\Delta O_L = \left(\frac{754}{3.2} + \frac{0.03 \times 5.25}{0.03+0.062-0.002} \right) \left(1 - e^{-0.002 \times 5} \right) - \left(\frac{0.03 \times 5.25}{0.03+0.062-0.002} \right)$$
$$\left(\frac{0.002}{0.062+0.03} \right) \left(1 - e^{-(0.062+0.03)5} \right) = 2.35$$

Then from Equation (V-40)

$$O_t = O_o - \Delta O_c - \Delta O_L$$

$$O_5 = 12.7 - \frac{3.32}{6.7} - 1.94 = 10.26$$

Solving the same equations with increasing t gives the data in Table V-15.

If it has been necessary to develop more data for the remainder of the stratified period, appropriately updated coefficients might be used starting at the beginning of each month.

TABLE V-15

DO SAG CURVE FOR QUIET LAKE HYPOLIMNION

Date	ΔO_L	ΔO_c	O_t
t = 0	0	0	0
5/5	2.35	1.30	9.05
5/10	4.68	2.13	5.89
5/15	6.99	2.65	3.06
5/20	9.22	2.98	0.50
5/25	11.54	3.18	0.00

Finally, if it is desired to evaluate the impact of altered BOD or phosphorus loadings, the user must go back to the appropriate step in the evaluation process and properly modify the loadings.

——————— END OF EXAMPLE V-15 ———————

5.6 APPLICATION OF METHODS AND PRESENTATION OF RESULTS: SOME RECOMMENDATIONS

This chapter has presented several approaches to evaluation of four impoundment problem areas. These are thermal stratification, sediment accumulation, eutrophication, and hypolimnion DO/BOD. In studying any or all of the potential problem areas in an impoundment, the user should first define the potential problems as clearly as he can. Often the nature of a problem will change entirely when its various facets are carefully described and examined *en masse*.

Once the decision is made that an aspect of impoundment water quality should be evaluated and the problem is clearly stated, the user should examine available solution techniques presented both in this document and elsewhere. The examination should address the questions of applicability, degree of accuracy, and need for data. The user will generally know what funds are available for data collection as well as the likelihood of procuring the needed data from previously developed bases. Also, the decision concerning needed accuracy rests with the user, and he should make decisions based upon the way in which his results will be used.

Once appropriate methods have been selected, the next task is to set down the data and to manipulate it according to computational requirements. Data are best displayed first in tabular form and then plotted in some meaningful way. Careful tabulation of data and graphing can themselves sometimes provide a solution to a problem, negating need for further analysis. Examples of plots of data include those of the thermal profiles in Figures V-1, V-2, and V-32 above. Where field observations of temperature are available, the data could well be plotted in this way. Another approach is to plot the difference between surface and bottom temperature over time.

Another example of data display is the sediment rating curve shown in Figure V-6. Plotting the mass discharge rate against flow for field data will produce a similar scatter plot. The degree of scatter is an indication of the utility of the data. If the data show high variability without an apparent trend, then the planner should collect more data or use an alternative approach. If the data appear to be well correlated, i.e., there is a clear trend and little scatter, a line-fitting procedure may be used to obtain an equation describing S_i as a function of flow.

Precipitation data may be displayed as hyetographs and long term plots showing annual trends. Runoff data may be plotted as runoff hydrographs (flow versus time) and tributary flows should similarly be plotted as a function of time. Pollutographs developed as concentration and as mass transport versus time should also be prepared for use in such methods as DO-BOD evaluation and assessment of eutrophication potential.

Following application of the methods discussed in this chapter, various graphical display methods may be useful for showing computational results. For example, in evaluating sediment accumulation in an impoundment, particularly where there is reason to believe that computations are fairly accurate, a longitudinal profile of the impoundment might provide the basis for a graphic display. The plot might show rates of deposition of various grain sizes as a function of time and distance through the impoundment. Accumulated sediment might be represented by lines drawn above the bottom with distance from the bottom of each line representing the magnitude of deposition. Figure V-33 is an example of this kind of display. Another alternative, might be to display the percent of inflowing sediment trapped as a function of grain size. This plot, an example of which is shown in Figure V-34, could easily contain curves for a number of impoundments within the study area.

Data on DO sag in the impoundment hypolimnion might be displayed graphically, although tabular results may be adequate. If a graphic display is desired, it can be done as in Figure V-35. An alternative might be to plot final DO (at overturn) as a function of loading. Loading can be represented as PO_4, algal biomass, or BOD.

In all cases, the user must determine how to best display results. A general guide is to prepare a graphic display wherever it will facilitate understanding of results, and the display should

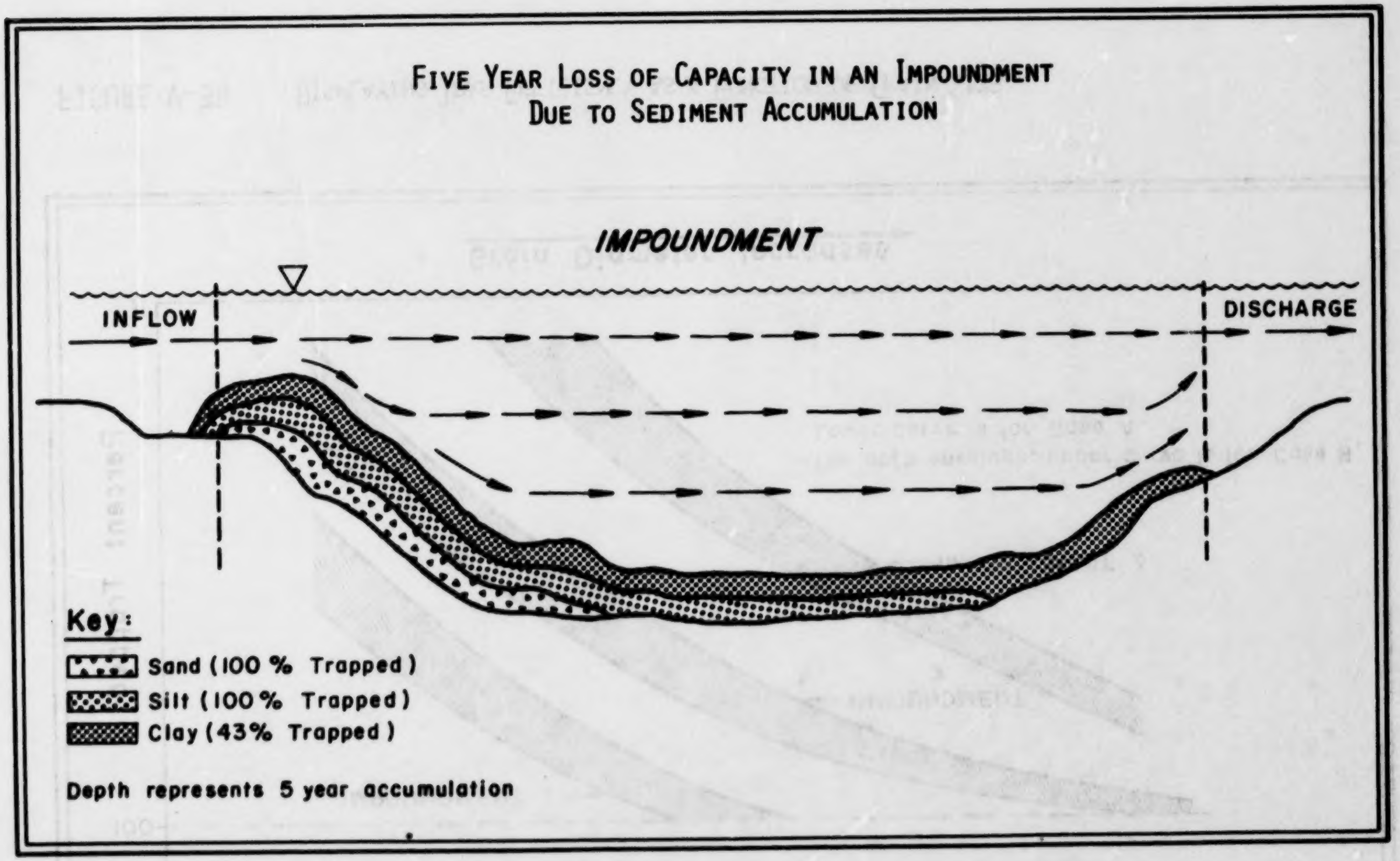
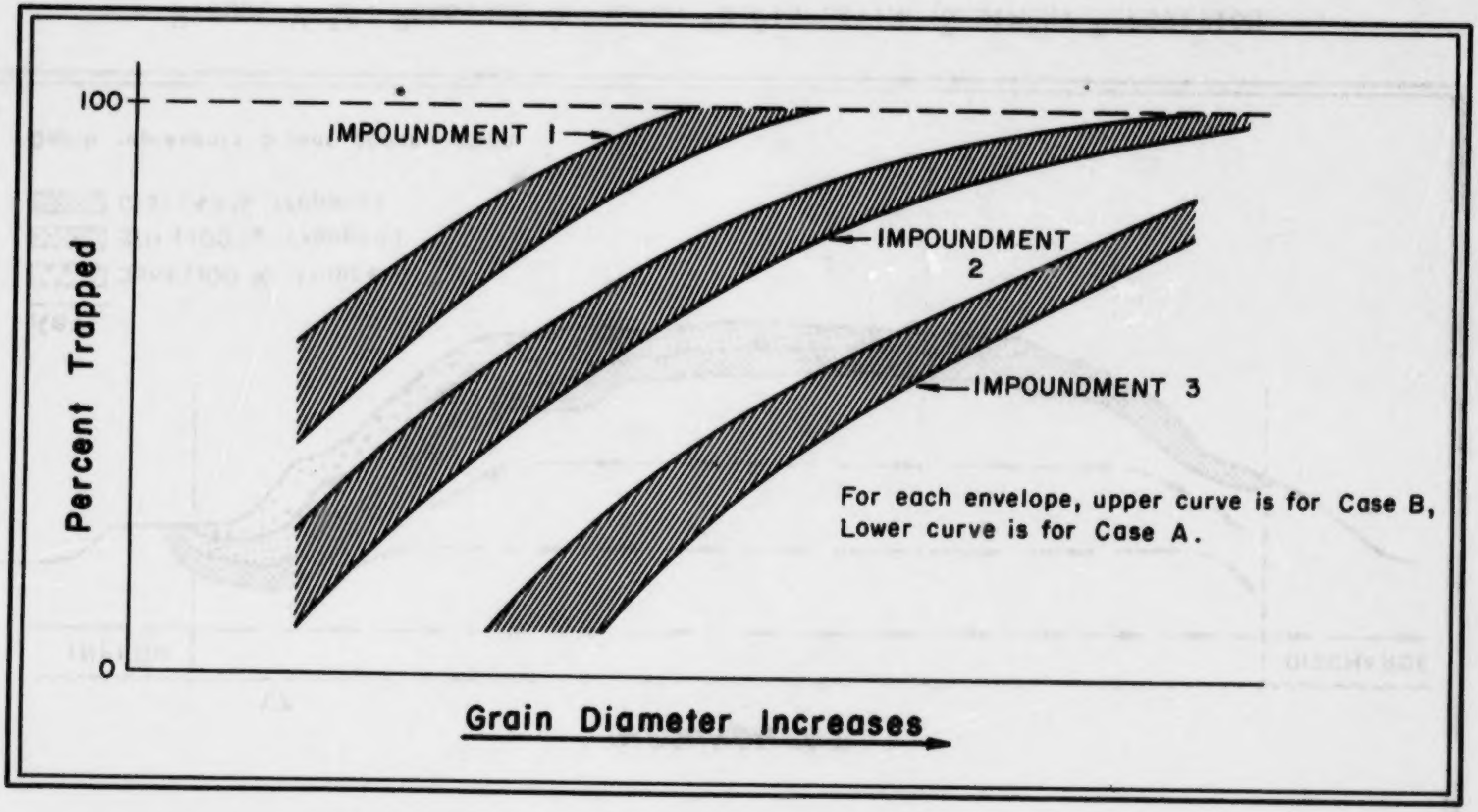

FIVE YEAR LOSS OF CAPACITY IN AN IMPOUNDMENT
DUE TO SEDIMENT ACCUMULATION
IMPOUNDMENT
INFLOW
DISCHARGE
Key:
Sand (100 % Trapped)
Silt (100 % Trapped)
Clay (43% Trapped)
Depth represents 5 year accumulation

FIGURE V-33 POSSIBLE APPROACH TO DISPLAYING SEDIMENT DEPOSITION RATES AS FUNCTION OF TIME AND DISTANCE DOWNSTREAM



For each envelope, upper curve is for Case B, Lower curve is for Case A.

FIGURE V-34 DISPLAYING TRAP EFFICIENCY AS A FUNCTION OF GRAIN SIZE

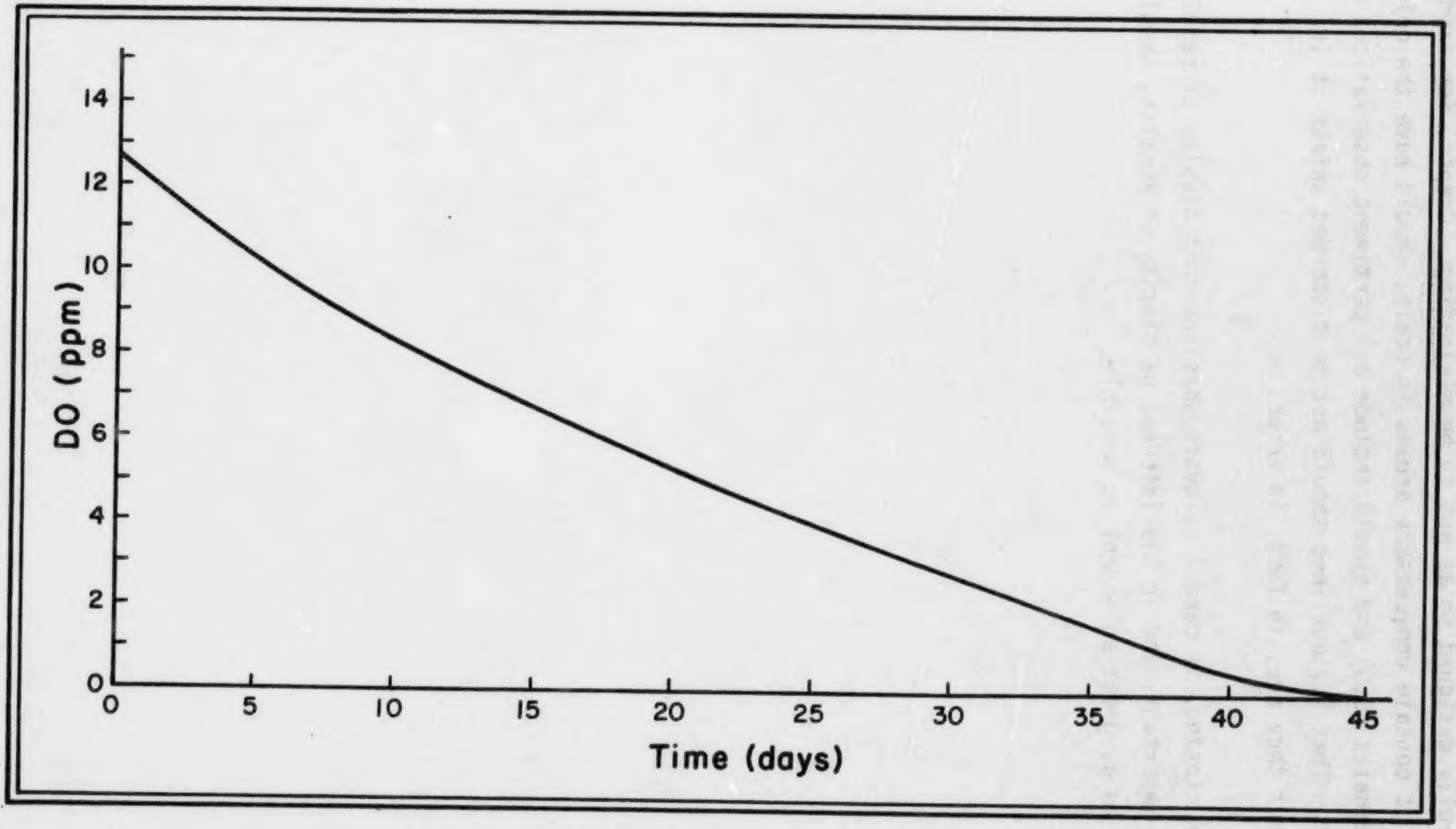


FIGURE V-35 AN EXAMPLE OF A PLOT OF DO VS. TIME IN IMPOUNDMENT HYPOLIMNION

be carefully designed so as not to be misleading. Graphs, for example, should not contain unnecessary breaks in scale, should have the origin at coordinates (0,0) and should include all pertinent observations of data set. That is, outlines should not be discarded unless it is clear that they are, in fact, in error.

In closing, it cannot be overemphasized that display of results is very important, and in the interest of clarity of results, should be used to as great an extent as possible.

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CHAPTER 6
ESTUARIES

6.1 INTRODUCTION

6.1.1 General

Estuaries are of primary social, economic, and ecologic importance to America. Forty-three of 110 of the Department of Commerce's Standard Metropolitan Statistical Areas are on estuaries (DeFalco, 1967). Estuaries are the terminal or transfer point for essentially all water-borne national and international commerce in this country, and biologically are more productive on a mass per unit area basis than any other type of water body. Essentially all conservative wastes and much of the nonconservative wastes discharged into any inland stream in America eventually pass into an estuary. Yet these coastal formations on which there is such a demand for services are less stable geologically than any other formation found on the continent (Schubel, 1971). Sedimentation processes, for example, are filling and destroying all estuaries. While this process is always rapid in a geological sense, the actual length of time required for complete estuarine sedimentation is a function primarily of the stability of the sea level, the rate of sediment influx, and the intra-estuarine circulation pattern (Schubel, 1971). The instability, variation, and complexity of estuaries make water quality assessment and prediction especially difficult, yet the demands placed on estuaries require a most active water quality management program.

This chapter will describe a systematic program which may be used to provide initial estuarine water quality assessment and predicition. Its purpose is two-fold. First, the planner will be provided the capability of making elementary assessments of current estuarine water quality. Second, a methodology is presented by which the planner can evaluate changes in water quality which might result from future changes in waste loading patterns.

Chapter 3 provided methodologies for assessing the waste load directly entering an estuary. Chapter 4 provided methodologies which can be used to assess the water quality of a river or stream as it enters an estuary. The output of this chapter will provide information about present and projected estuarine water quality which can be used to identify regions having greatest water quality problems, water quality parameters of special concern, and areas for which subsequent computer study is necessary. Methods presented below comprise a screening tool which may be used by the planner to focus attention on critical spatial regions and water quality parameters within a 208 area. These should then be more fully assessed using computer models or other sophisticated techniques, as required.

6.1.2 Estuarine Definition

It is difficult to provide a concise, comprehensive definition of an estuary. The basic elements included in most current definitions are that an estuary is:

a. a semi-enclosed coastal body of water,
b. freely connected to the open sea,
c. influenced by tidal action, and
d. a water body in which sea water is measurably diluted with fresh water derived from land drainage (Pritchard, 1967 and 1971).

The seaward end of an estuary is established by the requirement that an estuary be semi-enclosed. Because this boundary is normally defined by physical land features, it can be specifically identified.

The landward boundary is not as easily defined, however. Generally tidal influence in a river system extends further inland than does salt intrusion. Thus the estuary will be limited by the requirement that both salt and fresh water be measurable present. Accordingly, the landward boundary my be defined as the furthest measurable inland penetration of sea salts. The location of this inland boundary will vary substantially from season to season as a function of stream flows and stream velocities and may be many miles upstream from the estuarine mouth (approximately 40 miles upstream on the Potomac River, 27 miles on the James River, and approximately 16 miles upstream for the small Alsea Estuary in Oregon) (Pritchard, 1971). This definition also separates estuaries from coastal bays by the requirement for a fresh water inflow and measurable sea water dilution.

6.1.3 Types of Estuaries

While the above definition provides adequate criteria for segregating estuaries from other major types of water bodies, it does not provide a means to separate the various types of estuaries from one another. The variations in estuarine circulation patterns and resulting variations in pollutant dispersion from estuarine type to type make such a separation of estuaries a necessary part of any water quality assessment. Two basic estuarine classification systems have been used in recent years to accomplish estuarine subclass separation: a topographical system and a physical processes system (Dyer, 1973, Chapter 2 or Ippen, 1966, Chapter 10).

6.1.3.1 Topographical Classification

Under a topographical system, estuaries are divided into four subclasses. These are briefly described below.

a. Drowned River Valley (Coastal Plain Estuary). These estuaries are the result of a recent (within the last 10,000 years) sea level rise which has kept ahead of sedimentation processes at a river's mouth. Such estuaries are, quite literally, rivers whose lower basins have been drowned by the rising oceans. Coastal plain estuaries are characteristically broad, relatively shallow estuaries (rarely over 30 meters deep) with extensive layers of recent sediment.

b. Fjord-like Estuaries. These estuaries are usually glacially formed estuaries which are extremely deep (up to 800 m) but have shallow sills at the estuarine mouth. Fjord-like estuaries are restricted to high latitude mountainous regions and are not found in the United States outside of Alaska and Puget Sound in Washington.

c. Bar-built Estuaries. When offshore barrier sand islands build above sea level and form a chain between headlands broken by one or more inlets, a bar-built estuary is formed. These estuaries are characteristically very shallow, elongated parallel to the coast, and frequently are fed by more than one river system. As a result bar-built estuaries are usually very complex hydrodynamically. A number of examples of bar-built estuaries can be found along the southeast coast of the United States.

d. Tectonic Process estuaries. Tectonic estuaries exist as the result of major tetonic events (tectonic plates with associated faulting or subsidence and coastal volcanic activity). San Francisco Bay is a good example of an American estuary of this type.

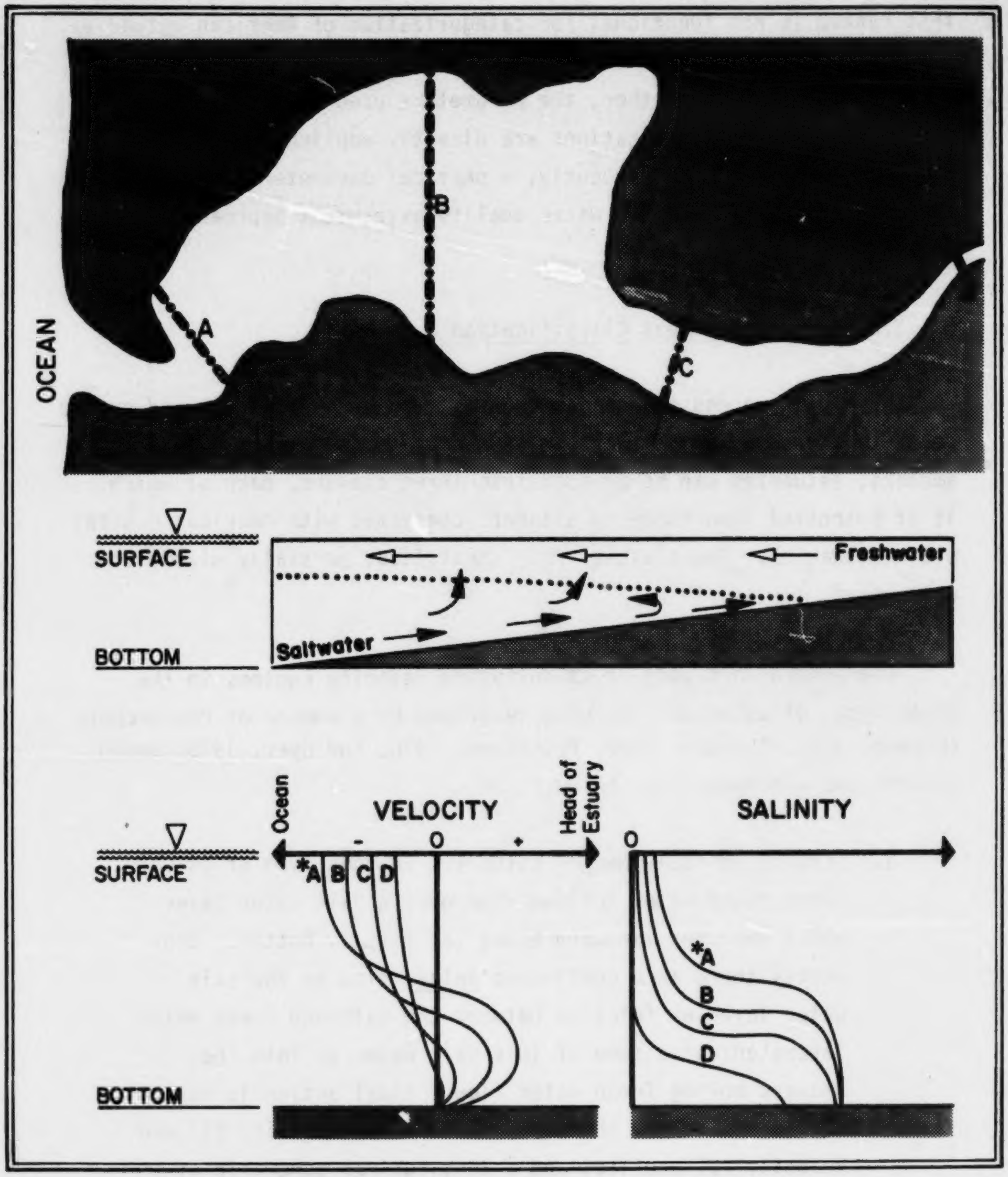
Based on this topographic classification system, the vast majority of American estuaries fall into the drowned river class. As a result, this system is not functional for categorization of American estuaries. The classification system based on physical processes is more useful, as described below. Further, the parameters used in calculating physical process classifications are directly applicable to estuarine pollution analysis. Consequently, a physical parameter classification system will be used for the water quality assessment approach to be presented.

6.1.3.2 Physical Process Classification

Physical process classification systems are generally based on the velocity and salinity patterns in an estuary. Using these two parameters, estuaries can be divided into three classes, each of which is of potential importance to planners concerned with American coastal plain estuaries. The classes are: stratified, partially mixed, and well mixed.

The general behavior of salinity and velocity regimes in the three types of estuaries has been described by a number of researchers (Glenne, 1967, Duxbury, 1970, Pritchard, 1970, and Dyer, 1973, among others) and are summarized below.

a. Stratified (Salt Wedge) Estuary. In this type of estuary, large fresh water inflows ride over a salt water layer which intrudes landward along the estuary bottom. Generally there is a continuous inland flow in the salt water layer as friction between the salt and fresh water layers entrains some of this salt water up into the seaward moving fresh water flow. Tidal action is not sufficient to mix the separate layers. Salinity (S) and Velocity (U) profiles and a longitudinal schematic of this flow pattern are shown in Figure VI-1. As an example of this type, the Mississippi River Estuary is usually a salt wedge estuary.


OCEAN
A
B
C
SURFACE
BOTTOM
Freshwater
Saltwater
SURFACE
BOTTOM
Ocean
VELOCITY
Head of Estuary
-
0
+
*A B C D
SALINITY
0
*A
B
C
D

*Letter correspond to cross section

FIGURE VI-1 TYPICAL MAIN CHANNEL SALINITY AND VELOCITY PROFILES FOR STRATIFIED ESTUARIES

b. Well Mixed. In a well mixed estuary, the tidal flow (or the tidal prism*) is much greater than the river outflow. Tidal mixing forces create a vertically well mixed water column with flow reversing from ebb to flood at all depths. Typical salinity and velocity profiles and a longitudinal flow schematic for a well mixed estuary are shown in Figure VI-2. As examples, the Delaware and Raritan River estuaries are both normally well mixed.

c. Partially Well Mixed. Partially well mixed estuaries lie between stratified and well mixed in terms of flow and stratification characteristics. Tide-related flows in such estuaries are substantially greater than river flows. Significant salinity gradients exist as in fully stratified estuaries, but are much less steep. While velocity at all depths normally reverses with ebb and flood tide stages, it is possible for net inward flow to be maintained in extreme lower layers. Typical salinity and velocity profiles and a longitudinal schematic flow diagram are shown in Figure VI-3. There are many partially mixed coastal plain estuaries in the United States, the James River Estuary being typical.

Classification primarily depends on the river discharge at the time of classification. Large river flows tend to result in more stratified estuaries while low flow conditions in the same estuary can lead to full mixing. Thus the classification of any single estuary is likely to vary from season to season as river flows vary. As examples, many west coast estuaries are partially mixed in winter when river flows are high and are well mixed in summer when river flows are very low.

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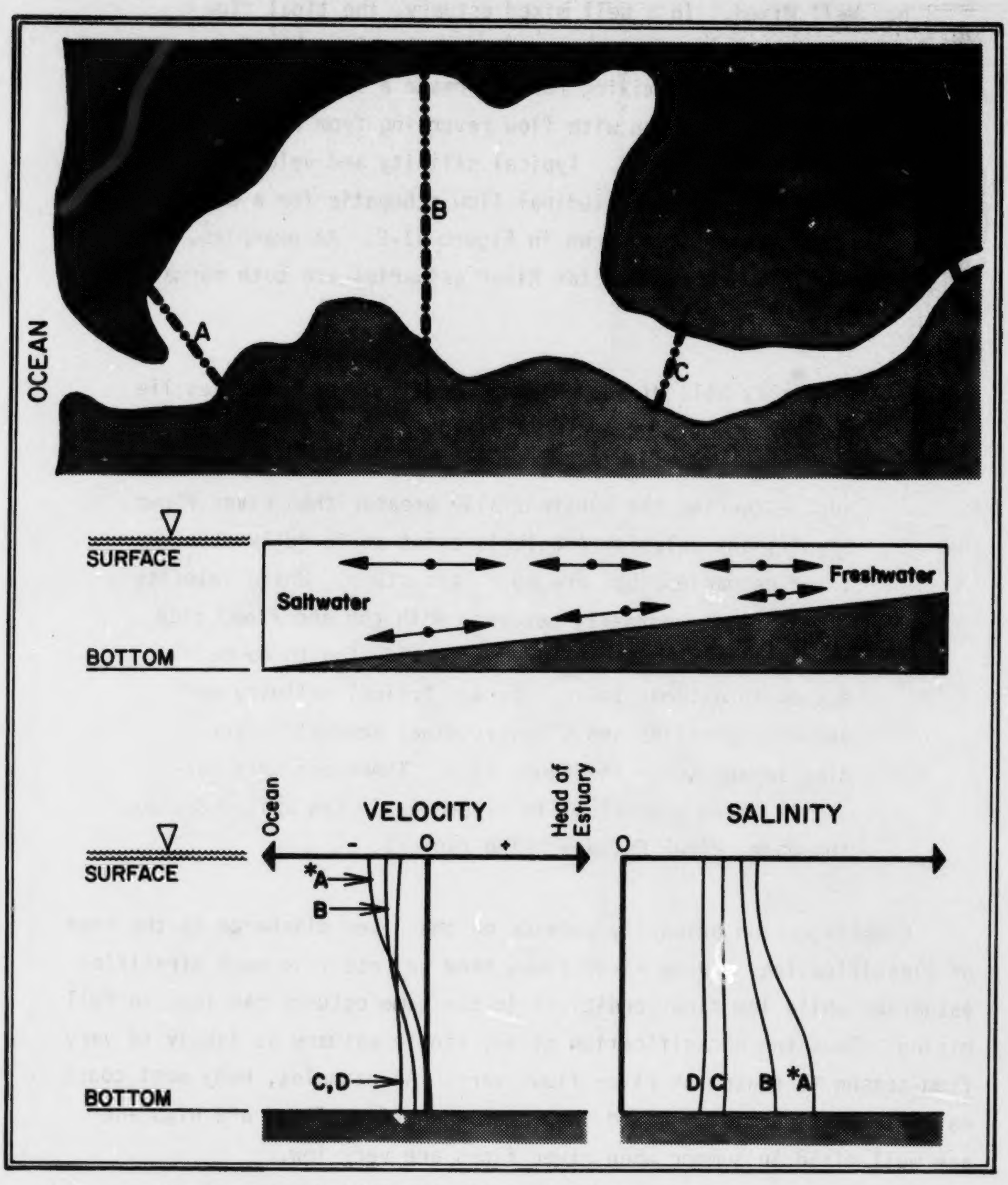
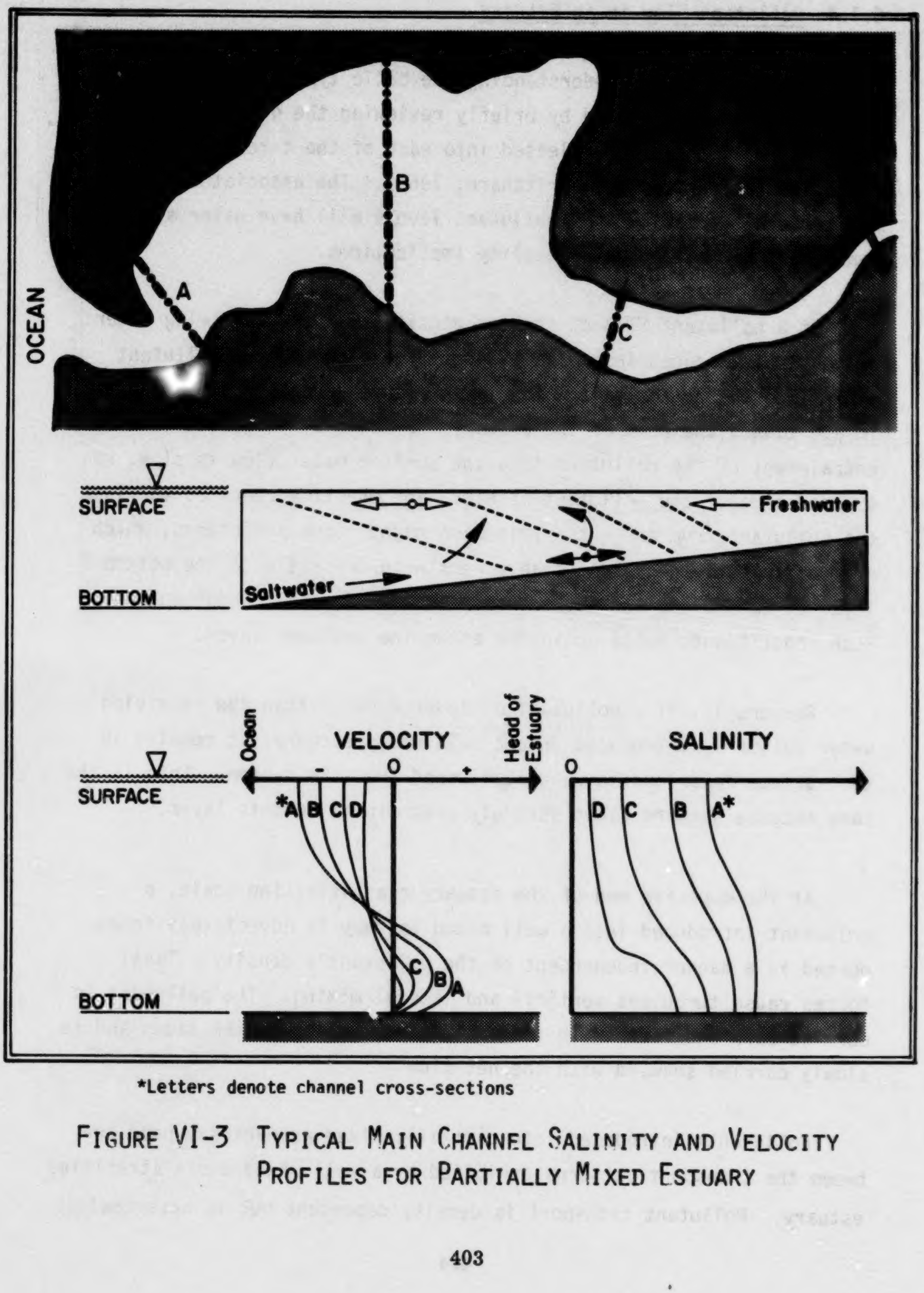
*The tidal prism is that volume of water which enters an estuary during an incoming (flood) tide and equals high tide estuarine volume minus low tide volume.


OCEAN
A
B
C
SURFACE
BOTTOM
Saltwater
Freshwater
Ocean
VELOCITY
Head of Estuary
SALINITY
-
O
+
O
SURFACE
*A
B
C,D
BOTTOM
D C B *A

*Letters correspond to channel cross-sections.

FIGURE VI-2 TYPICAL MAIN CHANNEL SALINITY AND VELOCITY PROFILES FOR WELL MIXED ESTUARIES


OCEAN
A
B
C
SURFACE
BOTTOM
Freshwater
Saltwater
Ocean
VELOCITY
Head of Estuary
-
O
+
SURFACE
*A B C D
C
B
A
BOTTOM
SALINITY
O
D C B A*

*Letters denote channel cross-sections

FIGURE VI-3 TYPICAL MAIN CHANNEL SALINITY AND VELOCITY PROFILES FOR PARTIALLY MIXED ESTUARY

6.1.4 Pollutant Flow in an Estuary

The importance of understanding the basic types of estuarine systems may be appreciated by briefly reviewing the general advective movements of a pollutant released into each of the three types of estuaries (summarized from Pritchard, 1960). The associated spatial and temporal variability of pollutant levels will have water system management as well as water quality implications.

If a pollutant flow of greater density than the receiving water column is introduced into a salt wedge type estuary, the pollutant sinks into the bottom salt water layer and is advectively carried inland toward the head of the estuary. Frictionally induced vertical entrainment of the pollutant into the surface water flow is slow, residence time of the pollutant is high, and the time required to flush the pollutant from the estuary is also high. Some pollutants, which are sufficiently dense and stable remain in or settle to the bottom layer of water, and are not transported out of a salt wedge estuary. Such constituents build up in the estuarine sediment layer.

Conversely, if a pollutant of lower density than the receiving water column is introduced into a salt wedge estuary, it remains in the surface layer and is readily flushed from the system. This is the case because seaward flows strongly predominate in this layer.

At the opposite end of the estuary classification scale, a pollutant introduced into a well mixed estuary is advectively transported in a manner independent of the pollutant's density. Tidal forces cause turbulent vertical and lateral mixing. The pollutant is carried back and forth with the oscillatory motion of the tides and is slowly carried seaward with the net flow.

Pollutants introduced into partially mixed estuaries behave between the transport patterns exhibited by a well mixed and a stratified estuary. Pollutant transport is density dependent but is nevertheless subject to considerable vertical mixing. Eventual flushing of the pollutant from an estuary in this case is dependent on the relative magnitude of the net river outflow.

6.1.5 Estuarine Complexity and Major Forces

Before outlining the complexities of estuarine systems, a brief review of the estuarine dimensions and nomenclature as used in this chapter will be helpful. This information is shown in Figure VI-4. This figure shows top, side, and cross sectional views of an estuary and indicates the basic estuarine dimensions. Additionally, the relationship between tidal elevation (or tidal stage) and surface water velocity is shown in the upper right quadrant of Figure VI-4.

The complexities of estuarine hydrodynamics are evident from even the brief qualitative descriptions presented above. Many variations in flow pattern and many of the forces acting on an estuarine water column have been omitted in order to permit a verbal description of the normally predominant phenomena, and it should be understood that the descriptions do not fully account for the complexities of estuarine motion. Estuarine circulation may be conceived as a three-dimensional flow field with independent variations possible in salinity and velocity along the longitudinal, the vertical, and the lateral axes. As a result of this complexity, and because an estuary is a transitional zone between fresh water and marine systems, great variations in a number of major water quality and physical parameters are possible. For example:

a. pH. Typical ocean pH is 7.8 to 8.4. Typically, rivers are slightly acidic (pH<7). Thus the pH changes from acidic to basic across an estuary with resulting major changes in chemical characteristics of dissolved and suspended constituents. Estuarine pH variations from 6.8 to 9.25 have been recorded (Perkins, 1974, p. 29)

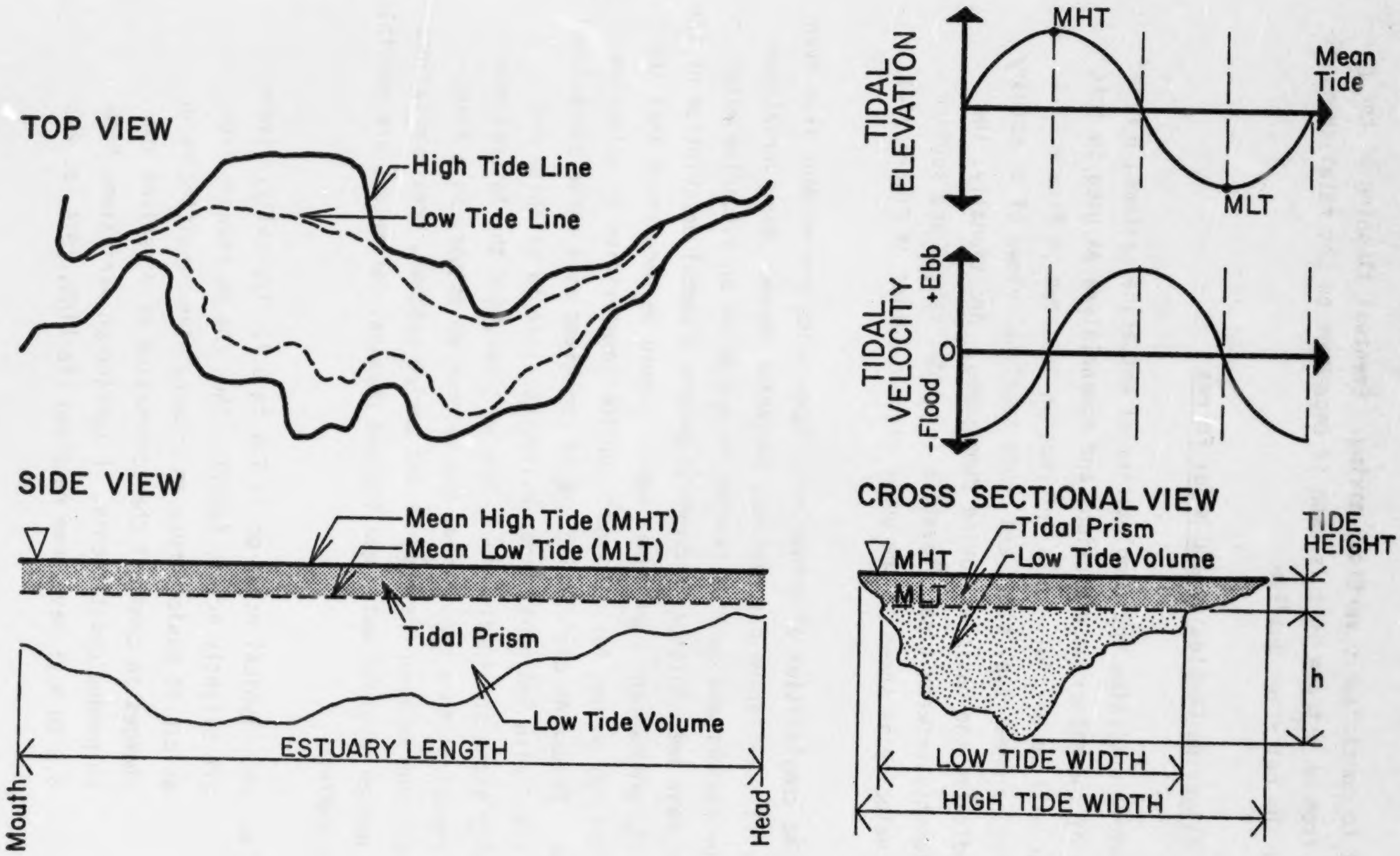

TOP VIEW
High Tide Line
Low Tide Line
SIDE VIEW
Mean High Tide (MHT)
Mean Low Tide (MLT)
Tidal Prism
Low Tide Volume
ESTUARY LENGTH
Mouth
Head
TIDAL ELEVATION
MHT
MLT
Mean Tide
TIDAL VELOCITY
+Ebb
0
-Flood
CROSS SECTIONAL VIEW
Tidal Prism
Low Tide Volume
MHT
MLT
TIDE HEIGHT
h
LOW TIDE WIDTH
HIGH TIDE WIDTH

FIGURE VI-4 ESTUARINE DIMENSIONAL DEFINITION

with surface waters commonly having a higher pH than bottom waters.

b. Salinity. Over the length of an estuary, salinity varies from fresh water levels (typically less than 1 ppt) to oceanic salinity levels (32 ppt to 34 ppt)*. Moreover salinity at any given location in an estuary may vary substantially over one tidal cycle and over the depth of the water column at any single point in time. Salinity variations are especially significant in estuarine calculations for a variety of reasons. First, salinity distribution can be used to predict the distribution of conservative pollutants; second, salinity levels are a prime determinant of water density; and third, variations in salinity affect other major water quality parameters. As an example of this last point, Figure VI-5 shows the effect of salinity variations on oxygen saturation concentration for a range of water column temperatures.

c. River Flow. River flow is a major determinant of estuarine circulation and flushing characteristics. Instantaneous flow rates for some western rivers vary by orders of magnitude from winter high flow to summer low flow periods (Goodwin, et al. 1970). These differences in river flow result in major variations in estuarine water quality characteristics.

d. Time. Estuarine water quality parameters vary over several separate time scales. First, variations occur with each tidal cycle over a period of hours. Second, tidal cycles vary in mean amplitude from spring (maximum amplitude) to neap tides (minimum amplitude) every two weeks. This affects water quality since flushing characteristics are in part dependent on the tidal prism which is, in turn, dependent on tide stage. Third, there are seasonal variations in river flow, temperature and waste loadings.

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*ppt represents parts per thousand by mass. Sometimes the symbol $^{0}/_{00}$ is used.

The four factors just listed affecting the range and rate of variation of estuarine parameters pose part of the difficulty in analyzing estuarine water quality. In order to avoid large errors, both small time increments and small spatial increments must be used.

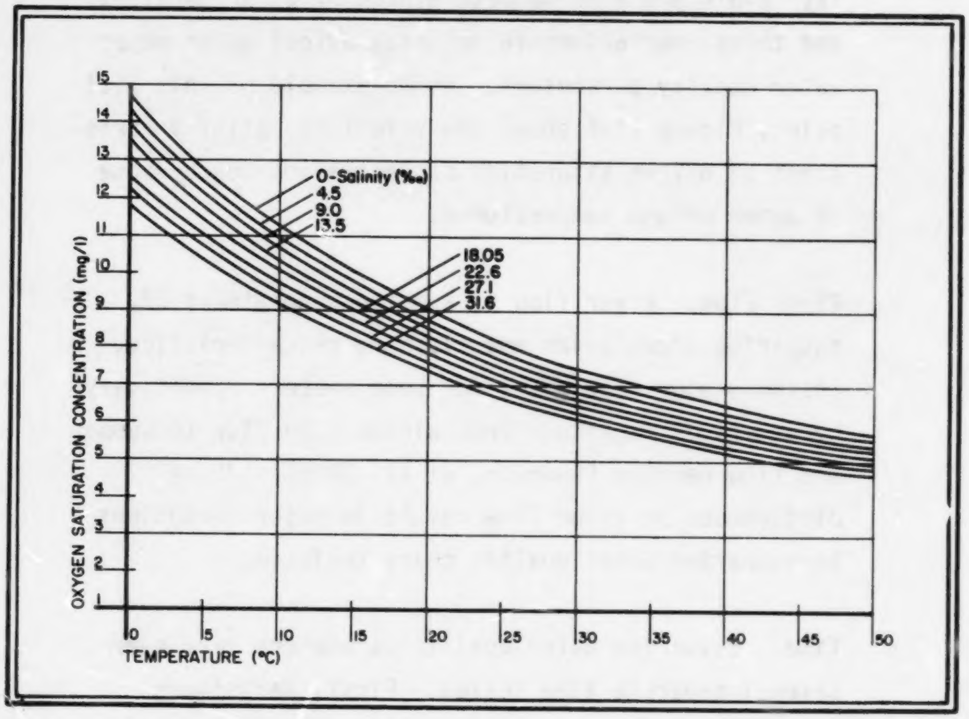

FIGURE VI-5 SATURATION TEMPERATURE SALINITY RELATIONSHIPS (GREEN, 1963)

This, in turn, necessitates a large number of individual calculations to fully analyze the variation of even a single parameter over the estuary and normally requires the use of computer model.

Further complicating the analytical process is the large number of independent forces acting on the estuarine water column which should be considered. This group of forces includes (from Harleman and Lee, 1969):

a. Ocean tides
b. Local wind stresses
c. Bottom roughness and bottom sediment types
d. Channel geometry
e. Coriolis forces*
f. Nearby coastal features and coastal processes

6.1.6 Simplifying Assumptions

A number of assumptions can be made which reduce the very complex problem of estuarine hydraulics to something amenable to hand calculations. Use of these assumptions results in a reduction in the accuracy of the analysis, a loss of spatially and temporally specific results (results will be averaged over large time segments and over large areas of an estuary), and an equating of the number of estuaries for which this methodology is valid to those meeting certain qualifying criteria. However, for those estuaries meeting the qualifications, the methodology will provide a valuable estimate for a number of water quality parameters.

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* Coriolis forces relate to the effect of a rotating reference plane (the earth) on particle motion. The net effect is to cause a water flow to drift to one side as it propagates down a channel. The same effect tends to laterally segregate fresh water flows (moving from head to mouth) and salt water inflows (moving from mouth to head) in an estuary and in the northern hemisphere to create a counterclock-

The basic assumptions involved in the simplifying process are:

a. Present salinity distribution can be used as a direct measure of the distribution of all conservative, continuous flow pollutants entering the estuary, and can be used as the basis for calculating dispersion coefficients for one-time discharges.

b. The vertical water column is assumed to be homogenous. Calculations will be based on vertically well mixed estuaries, but can be empirically modified to account for the partially mixed situation. However, the complexity and multi-dimensional nature of stratified circulation excludes consideration of these estuaries.

c. All estuaries are assumed to be one-dimensional. This assumption is the implicit result of assumptions b and c, which assume lateral and vertical homogeneity.

d. Coriolis forces may be ignored. This means that an estuary may be assumed to be laterally homogeneous through any cross-section and typically excludes consideration of very wide, slow moving estuaries since the significance of the Coriolis forces is inversely proportion to characteristic estuarine water velocities. Dr. Ralph Cheng (1977) of the United States Geological Survey has offered a criterion for checking compliance with

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wise flow pattern with fresh water to the right (looking from the head of the estuary toward the mouth) flowing toward the sea and salt water on the left flowing toward the head of the estuary.

this assumption using the Rossby number (R_o)*. For R_o values less than 0.1, Coriolis forces will make a relatively small contribution to water mass movements. Some error, however, will be introduced for R_o values above 0.02 to 0.03. For the level of calculation considered in this report it is recommended that Coriolis forces be ignored. This assumption is especially applicable to small U.S. estuaries. However, it must be rememberd that the wider the estuary, and especially the lower its range of typical velocities, the greater the error introduced by ignoring Coriolis forces.

e. Only steady-state conditions will be considered. Calculations will provide values averaged over a tidal cycle**. Further, it will be assumed that for continuous flows, only the final steady-state distribution of a pollutant is of concern to the planner.

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*The Rossby number is defined by (Neumann and Pierson, 1966)

$$R_o = \frac{\text{characteristic velocity}}{\Omega L}$$

where: characteristic velocity = "typical water velocity". This may be aproximated by 1/2 of the peak surface velocity,

L = the length of the estuary,

Ω = earth's rotational rate at the latitude of the estuary (earth's rotational rate at the equator multiplied by the sine of the latitude).

**This type of calculation will be called a "net non-tidal" calculation and provides only the distribution of a pollutant mass at the end of successive tidal cycles. It does not track variations in its distribution during the tidal cycle from, for example, low slack water to high slack water and back to low.

f. Wind forces can be ignored. For large, shallow estuaries (for example, most bar-built estuaries) wind forces are a major mechanism for both lateral and vertical mixing. The inclusion of wind, however, cannot be readily accommodated in hand calculations. Further, under all but strong and severe wind conditions, wind is not a major controlling factor in steady state conditions.

g. Regular geometry is assumed. It is assumed that, at least over the length of each segment, the estuary geometry is regular and can be approximated by a constant cross-section. This means that the calculations are not valid for many bar-built, fjord-type, or complex estuaries with large side embayments.

h. Only one river inflow is allowed. Estuaries with two significant fresh water inflows have complex hydrodynamic characteristics, and for these estuaries, computer models should be used. However, when one fresh water flow is significantly larger than all others, the dominant river may be considered to be the only river inflow. All others may be considered as point source pollutant inflows.

i. No variations in tidal amplitude will be considered. In many areas of the U.S., the two high and two low tides occurring each day are not equal. As a rule, they go from lower low, to higher high, to higher low, to lower high, and back to lower low (Ippen, 1966). Further, higher high tides vary significantly from spring to neap tide. For purposes of these calculations all tidal cycles are assumed of equal amplitude.

j. All water leaving the estuary each tidal cycle is replaced by "fresh" sea water. This assumes that none of the pollutant flushed out of an estuary with one ebb tide will re-enter the estuary on the subsequent flood tide. This approximation can be accounted for in flushing time calculations by reducing the total tidal flow to an "effective" tidal flow if specific conditions indicate that a significant percent of tidal water returns into the estuary (see Section 6.4).

6.1.7 Methodology Summary

The techniques presented in this chapter are based on past investigative work. While there is little in terms of new technique presented below, this chapter represents a compilation of all major methodologies applicable to the hand calculation process. There are techniques more sophisticated than those presented here. However, it is felt that such techniques are either too cumbersome or require too much input information to be amenable to hand calculations.

This chapter constitutes a step-wise methodology. While the goals and techniques of each step are separately discussed in the appropriate sections, a summary of the goals of each step and their sequential order is given here.

Step	Goal
1. Existing Water Quality Assessment · Estimate present estuarine water quality	Determine which estuaries have most severe water quality problems; obtain estimate of current "worst-case" estuarine water quality.
2. Future Water Quality Assessment	
· Classify Estuary	Determine whether hand calculations are appropriate for a given estuary
· Estimate Flushing Time	Determine pollutant residence time in the estuary; improve estimate of pollutant concentration

· Calculate New Source Pollutant Distribution	Estimate the distribution of pollutants associated with additional waste loadings; estimate resulting changes in estuarine water quality
· Calculate Waste Heat Distribution	Estimate distribution of excess heat resulting from a single thermal discharge
· Estimate Sediment Transport and Deposition	Estimate suspended sediment and bed load movement through an estuary; identify areas of critical sediment deposition, and assess estuarine turbidity.

An overall schematic diagram for the procedures for the predictive portion of the estuarine hand calculation methodology is shown in Figure VI-6. The water quality parameters which will be addressed in each major step of the analytical process are shown in the left hand column of that figure. More detailed schematics for the various components shown are contained in each associated section of this report. Note from Figure VI-6 that the hand calculations do not represent a definitive final analysis of estuarine water quality. The results obtained here will serve only as a screening process to identify areas and individual estuaries for which application of computer models is indicated and will allow the planner to set analytical priorities.

6.1.8 Data Requirements

One consideration in the selection of the techniques presented in this report was a minimization of data requirements. Still, certain data will be needed for each estuary in order to use any of the calculations. These data generally include:

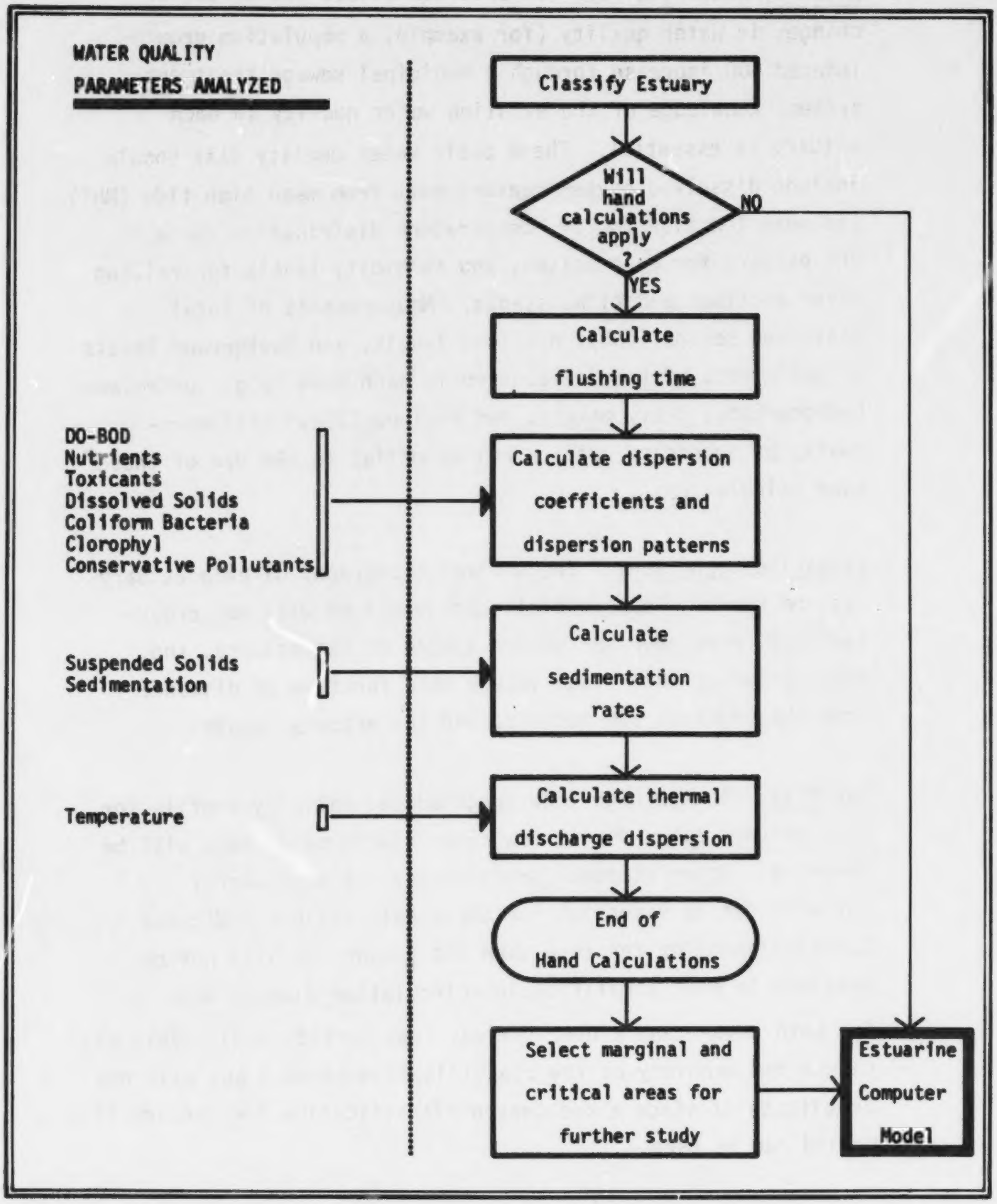
WATER QUALITY
PARAMETERS ANALYZED
Classify Estuary
Will hand calculations apply ?
NO
YES
Calculate flushing time
DO-BOD
Nutrients
Toxicants
Dissolved Solids
Coliform Bacteria
Clorophyl
Conservative Pollutants
Calculate dispersion coefficients and dispersion patterns
Suspended Solids
Sedimentation
Calculate sedimentation rates
Temperature
Calculate thermal discharge dispersion
End of Hand Calculations
Select marginal and critical areas for further study
Estuarine Computer Model

FIGURE VI-6 DIAGRAM OF ESTUARINE WATER QUALITY CALCULATION PROCESS

- Existing water quality: In order to assess the effect of changes in water quality (for example, a population growth-induced BOD increase through a municipal sewage treatment system) knowledge of the existing water quality in each estuary is essential. These basic water quality data should include dissolved oxygen measurements from mean high tide (MHT) and mean low tide (MLT)*, temperature distribution through the estuary for each season, and turbidity levels for raising river outflows and tidal stages. Measurements of total dissolved solids (TDS), nutrient levels, and background levels of pollutants of special concern in each area (e.g., petroleum hydrocarbons, heavy metals, and radionuclides) will eventually be necessary but are not essential to the use of these hand calculations.

- Estuarine topography: The general topography of each estuary must be known. Topographical data required will be: cross-sectional area over the entire length of the estuary, the tidal prism or intertidal volume as a function of distance from the mouth of the estuary, and the estuary length.

- Salinity: A mean high tide longitudinal salinity profile for each estuary taken during low river discharge periods will be required. Other seasonal profiles will be very useful but will not be essential for these calculations. Without salinity profiles for more than one season, it will not be possible to plot stratification-circulation diagram data for both summer and winter periods (see Section 6.3). This will reduce the accuracy of the classification process but will not invalidate it since a one season classification for the low flow period can be used.

---

*These terms as used here are synonymous with the terms "mean higher-high tide" (MHHT) and "mean lower-low tide" (MLLT).

- River discharge: River discharge rates into the estuary for annual low flow periods (normally measured in cubic feet per second (cfs)) will be required. Again river discharge rates throughout the year are desirable but not essential for these calculations.

- Velocity profiles: Finally, vertical velocity profiles* for several stations in each estuary will be needed for at least a one tidal cycle duration for both low and high river discharge periods. These stations should be within the main estuary channel and should include one station near the mouth and one well upstream toward the head of the estuary.

Further detail on these data parameters and some potential sources from which they may be obtained will be discussed in each section as appropriate.

6.2 PRESENT WATER QUALITY ASSESSMENT

The first step in the estuarine water quality assessment must be the evaluation of existing water quality. Before an analysis of the impact of future waste load changes is made, the planner should know whether or not current estuarine water quality is acceptable, marginal, or substandard.

By far the best way to assess existing water quality is to measure it. The planner should attempt to locate other agencies which might have already collected acceptable samples and/or data. Candidate organizations include the United States Geologic Survey, the U.S. Army Corps of Engineers, state water quality control and monitoring agencies, and Engineering and Oceanographic departments of local colleges and universities. If such data cannot be located, a data collection program sponsored by the 208 planning agency should be undertaken. If at all possible, high tide, and especially low tide in-situ measurements and samples should be collected along the full length of the estuary's main channel and in all significant side embayments. Analyses should then be made in an appropriate laboratory facility. If funds for such data collection effort are not available, the use of a mathematical estimation of existing water quality is a third alternative. One very coarse method for such water quality estimation is presented here. However, it should be remembered that actual data is always preferable to a mathematical estimate of water quality.

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* Surface and bottom velocities are acceptable if more detailed vertical profiles cannot be obtained.

This section will provide a method for estimating estuarine water quality and will be useful where data are lacking. Chapter 3 presented methods for evaluating the total direct waste load. Chapter 4 presented methods for evaluating river water quality (concentrations of pollutants in the river flow). Combining these allows evaluation of the total waste load enterine an estuary over any time period. This total load of each individual pollutant type (e.g., conservatives, BOD) may be converted into average pollutant concentration in the estuary through use of Equation VI-1 below.

$$c_i = \frac{F\left(\sum_{j=1}^{n} (Q_j c_{ij})\right)}{VOL} \qquad \text{(VI-1)}$$

where c_i = average concentration of pollutant "i" in the estuary, normally in mg/l

c_{ij} = concentration of pollutant "i" in waste load source "j," normally in mg/l

Q_j = flow rate of discharge "j" (normally obtainable in cubic feet per second (cfs) or in million gallons per day (mgd) and must be converted as shown below n = number of discrete waste loads discharging into the estuary

F = flushing time (normally measured in tidal cycles)

and VOL = water volume of the estuary at low tide (in cubic feet). Low tide volumes are often obtainable from the sources of potential estuarine water quality data previously discussed or may be estimated from U.S. Coast and Geodetic Survey coastal map series sheets.

In this equation, measurements of Q_j must be converted from cfs or mgd units to cubic feet per tidal cycle. Conversion from cfs units involves only a time conversion (3600 sec/hour x 12.4 hour/tidal cycle). From mgd units it involves both a time conversion (1 day/24 hr x 12.4 hr/tidal cycle) and a volumetric conversion (∿1 cubic ft/7.8 gal). Once the waste load is in units of cubic feet/tidal cycle, it may be multiplied by the flushing time to obtain a total estuarine mass of pollutant "i".

There are a number of methods available for estimation of flushing time. This term represents the residence time of a mass of water (or of a pollutant) within the estuary. In other words, the flushing time represents the amount of time required to replace all of the fresh water within the estuary by "new" water from the fresh water (river) inflow. Two methods of estimating flushing time are presented here. More explicit versions of both will be used later. (See section 6.4.5). The more general versions presented below are applicable for this level of analysis.

6.2.1 Estuarine Fraction of Fresh Water

This method requires average estuarine salinity data (averaged longitudinally over the length of the estuary and temporally over a tidal cycle). With this data, F is given by:

$$F = \frac{S_s - S_e}{S_s} \qquad \text{(VI-2a)}$$

where S_s = oceanic salinity (generally 32 ^{o}/oo to 36 ^{o}/oo depending on location)

S_e = mean estuarine salinity (in parts per thousand, ^{o}/oo)

and F = flushing time in tidal cycles

6.2.2 Tidal Prism Method

If salinity data are not available, flushing time may be estimated by

$$F = \frac{V + P}{P} \qquad \text{(VI-2b)}$$

where F = flushing time in tidal cycles,

V = low tide volume of the estuary,

and P = estuarine tidal prism (volume of water between low tide level and high tide level)

These procedures may be repeated for each pollutant parameter to obtain an overall assessment of estuarine water quality.

6.2.3 Interpretation of Results

The water quality data obtained by this analysis must be considered as a first estimate. It has provided only an overall relationship between the gross amount of pollutants entering an estuary and the water mass within the estuary. In addition to the assumptions discussed earlier, the following assumptions have been added to these calculations:

1) The estuary is assumed to be totally mixed.

2) All inflows are assumed to disperse throughout the estuary in exactly the same manner regardless of the discharge location. In fact, as will be shown in subsequent sections,

the location of discharge will have a major impact on both the magnitude and peak location of resulting pollutant concentration.

3) Pollutant concentrations are assumed constant over time (over a tidal cycle and from tidal cycle to cycle).

If these approximations result in unacceptably large uncertainties in current pollutant load estimation the more detailed methods presented in subsequent sections for the evaluation of changes in water quality may be applied to the evaluation of present water quality. This process requires the separate evaluation of each discrete discharge. To avoid excessive effort in such a calculation, adjacent discharges should be grouped wherever possible and treated as a single point source. Total estuarine concentrations of the various pollutants are then estimated by summing the steady state distributions of each discharge in order to obtain total concentration over the length of the estuary.

6.3 ESTUARINE CLASSIFICATION

6.3.1 General

The previous section presented a method for making a first estimate of current estuarine water quality when no actual data were available. This section begins a calculation methodology designed to look at the effect of future changes in waste loading patterns. Because such an analysis will focus on a limited number of discharges at once, the technique presented will be much more detailed and sophisticated than those of the previous section.

The goal of the classification process presented below is to predict the applicability of the hand calculations to be presented. As described earlier, a number of limiting assumptions have been made to reduce the complexity of computations. The classification process is the first step to be taken in the calculation procedure since it selects those estuaries for which techniques to be presented can be applied.

6.3.2 Classification Methodology

The classification system recommended for purposes of hand calculations is based on salinity and velocity profiles within the estuary. As both of these parameters vary seasonally and spatially for each estuary, their use will result in a range of values rather than in one single classification number.

A schematic of the classification process presented in this section is shown in Figure VI-7. The following section will describe in detail the procedure for use of this system, and show examples of the procedure.

6.3.3 Calculation Procedure

6.3.3.1 Geometric Pattern

This first step is a subjective one. All of the calculations used in this chapter assume a single "drowned river valley" estuarine geometry. The presence of major side embayments not along the central river channel or of more than one significant fresh water inflow violates this assumption and means that hand calculations should not be attempted for that estuary. As can be seen in Figures VI-1 through VI-3 this does not mean that the estuarine channel must be completely linear and uniform. However, the farther away from an idealized channel an estuary is, the larger will be the error in the final water quality estimations. Figure VI-8 shows examples of three estuaries for which one might proceed with these calculations, and two estuaries for which one should not.

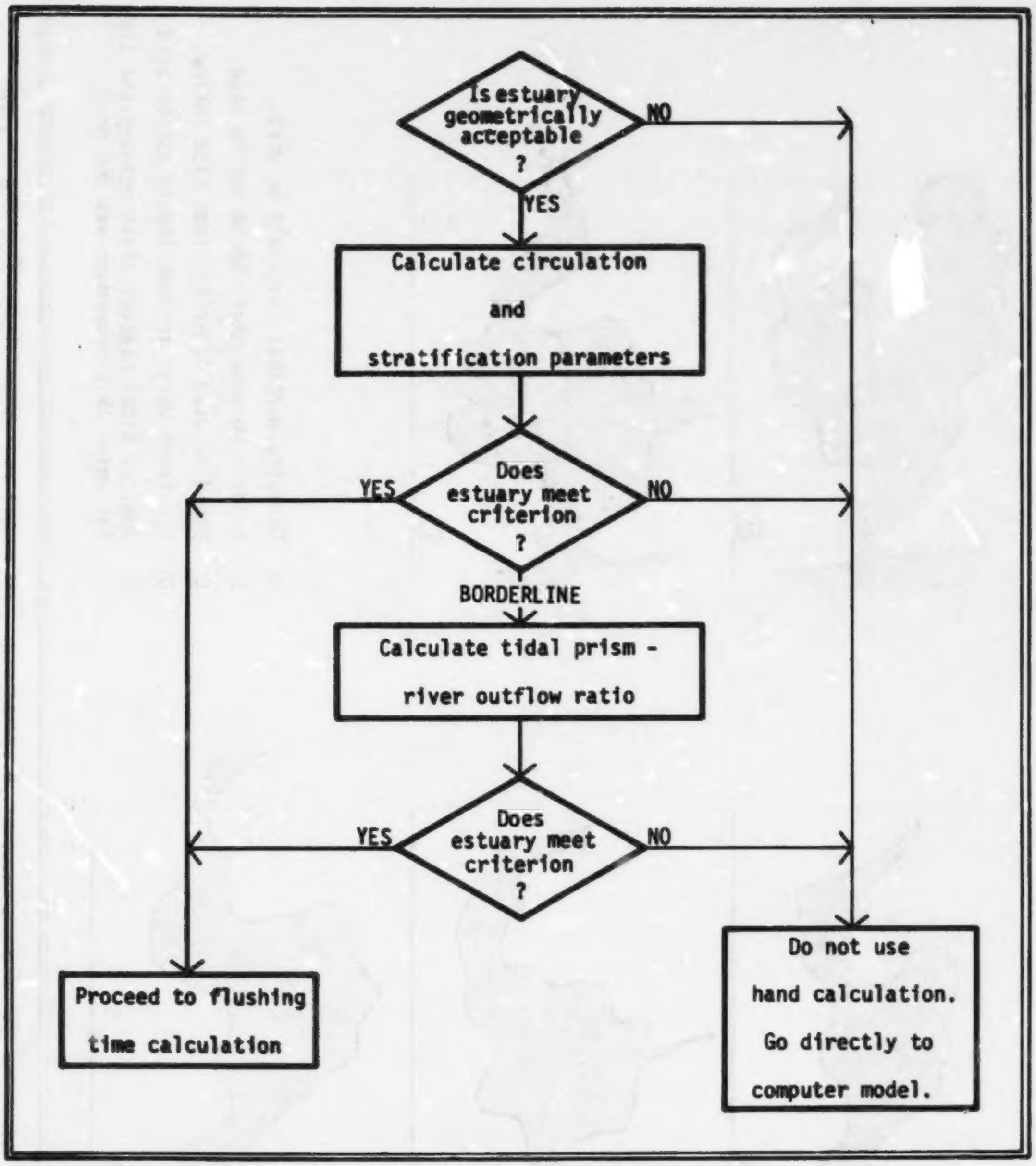
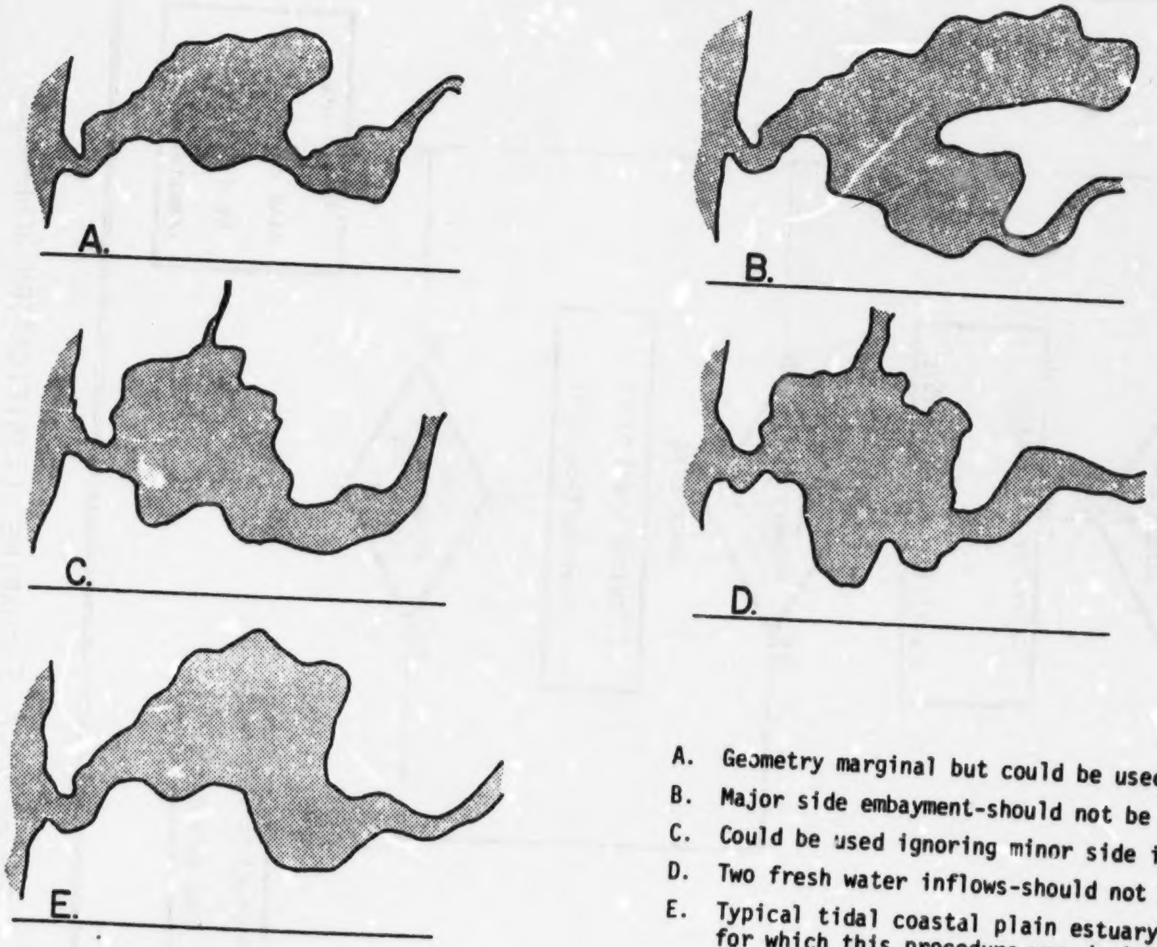

Is estuary geometrically acceptable ?
NO
YES
Calculate circulation and stratification parameters
Does estuary meet criterion ?
YES
NO
BORDERLINE
Calculate tidal prism - river outflow ratio
Does estuary meet criterion ?
YES
NO
Proceed to flushing time calculation
Do not use hand calculation. Go directly to computer model.

FIGURE VI-7 ESTUARINE CLASSIFICATION SCHEMATIC



A. Geometry marginal but could be used.
B. Major side embayment-should not be used.
C. Could be used ignoring minor side inflow.
D. Two fresh water inflows-should not be used.
E. Typical tidal coastal plain estuary-the type for which this procedure was designed.

FIGURE VI-8 EXAMPLES OF ESTUARINE SUITABILITY FOR HAND CALCULATIONS

6.3.3.2 Stratification-Circulation Diagram

Hansen and Rattray (1966) developed an estuarine classification system using both salinity stratification and water circulation patterns (based on water column velocities). This procedure involves the calculation of values for two parameters at various points along the main estuarine channel and the plotting of these intersections on the graph shown in Figure VI-9. Figure VI-10 shows plots made by Hansen and Rattray for various estuaries at a single point in time. It should be noted that each estuary is not represented by a point but by a line connecting the plots calculated for the mouth and head areas.

The area designations (e.g., 1a, 1b, 2a) on Figure VI-9 were related by Hansen and Rattray to previously used classification titles (e.g., stratified, well mixed). In general, area 1a corresponds to well mixed estuaries. Area 1b has the water circulation pattern of a well mixed estuary yet shows increased stratification. Areas 2 and 3 correspond to the "par[illegible]y mixed" class of estuaries with area 3 showing more significa[illegible]eral circulation within the estuary. Designations 2a/b and 3a/b, as was true of 1a and 1b, indicate increasing degrees of vertical stratification. Type 3b includes fjord-type estuaries. Area 4 represents highly stratified, salt wedge estuaries.

The two parameters used with this diagram are described below:

a. Stratification Parameter: The stratification parameter is defined as*:

$$\text{Stratification Parameter} \equiv \frac{\Delta S}{S_o} \qquad \text{(VI-3)}$$

---

*The symbol $\equiv$ as used in this chapter implies a definition.

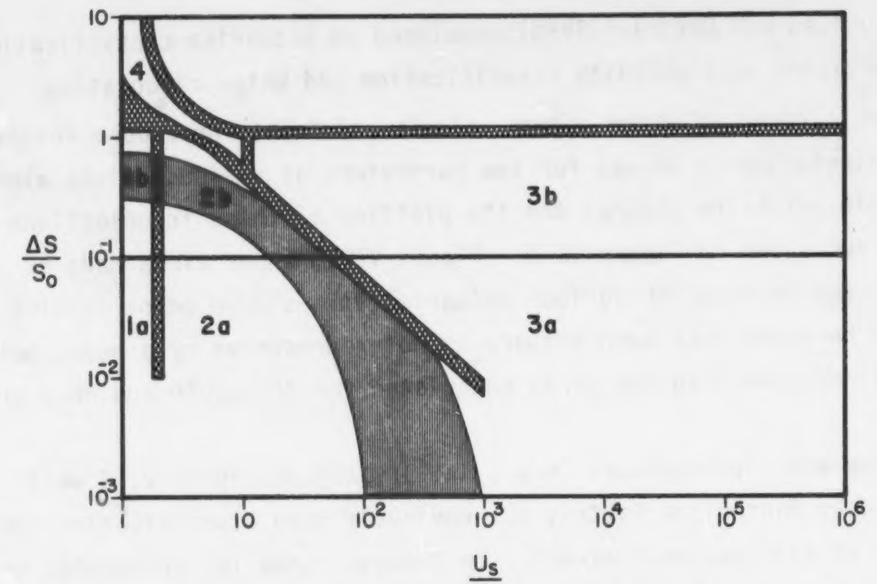
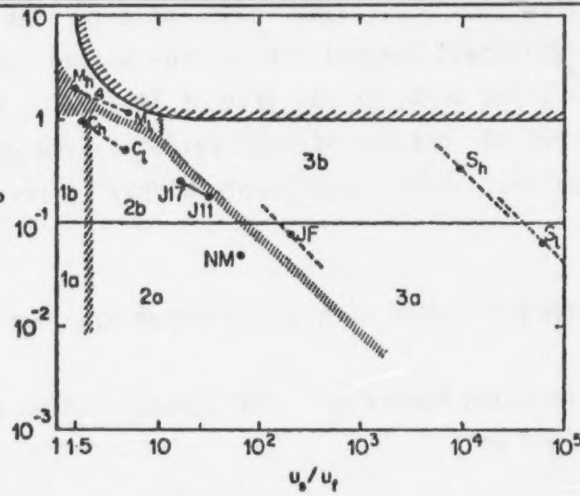


FIGURE VI-9 ESTUARINE CIRCULATION-STRATIFICATION DIAGRAM



(Station code: M, Mississippi River mouth; C, Columbia River estuary; J, James River estuary; NM, Narrows of the Mersey estuary; JF, Strait of Juan de Fuca; S, Silver Bay. Subscripts h and l refer to high and low river discharge; numbers indicate distance (in miles) from mouth of the James River estuary.

FIGURE VI-10 EXAMPLES OF ESTUARINE CLASSIFICATION PLOTS (FROM HANSEN AND RATTRAY, 1966)

where

ΔS = time averaged difference in salinity between surface and bottom water $(S_{bottom} - S_{surface})$

and,

S_o = cross-section mean salinity $\frac{(S_{bottom} + S_{surface})}{2}$

The diagramatic relationship of these values is shown in Figure VI-11.

b. Circulation Parameter: The circulation parameter is defined as:

$$\text{Circulation parameter} \equiv \frac{U_s}{U_f} \qquad \text{(VI-4)}$$

where

U_s = net non-tidal sectional surface velocity (surface velocity through the section averaged over a tidal cycle) measured in ft/sec. See Figure VI-11 for a diagramatic representation of U_s.

and

U_f = mean fresh water velocity through the section

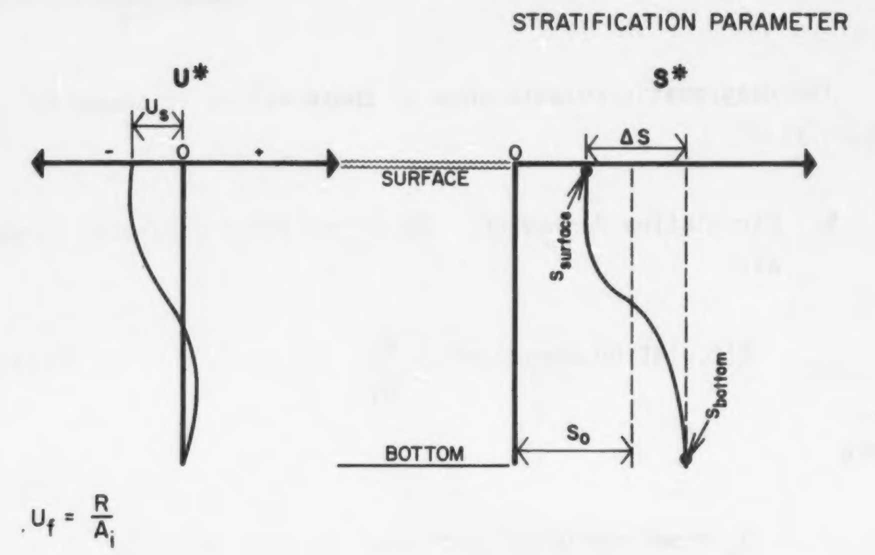
In equation form,

$$U_f = \frac{R}{A_i} \qquad \text{(VI-4a)}$$

where

R = fresh water (river) inflow rate measured in ft^3/sec,

and A_i = a cross-sectional area of the estuary through the point being used to calculate the circulation pattern and stratification parameters based on a mean tide surface elevation



* Both U and S values for these profiles are averaged over a tidal cycle (net velocity and salinity) rather than being instaneous values. Of the two the stratification parameter is much less sensitive to variations over a tidal cycle and can be approximated by mean tide values for salinity. Surface velocity (U_s) must be average over a tidal cycle.

FIGURE VI-11 CIRCULATION AND STRATIFICATION PARAMETER TERM DIAGRAM

If exact cross-sectional area data are not available, cross-sectional profiles can be approximated from the Coast and Geodetic Survey coastal series topographical maps, or, more recently, from NOAA National Ocean Survey map sheets.

These two parameters should be plotted for high and low river flow periods and for stations near the mouth and head of the estuary.

The area enclosed by these four points should then include the full range of possible instantaneous estuary hydrodynamics characteristics. In interpreting the significance of this plotted area, by far the greater weight should be given to the low river flow periods as these periods are associated with the poorest pollutant flushing characteristics and lowest estuarine water quality. The interpretation of the circulation-stratification diagrams will be explained more fully after an example of parameter computation.

EXAMPLE VI-1

Calculation of Stratification and Circulation Parameters

The estuary for this example is the Typical Estuary which is shown in Figure VI-12. The estuary is 64,000 feet long, is located on the U.S. west coast, and is fed by the Running River. Two stations were selected for parameter calculation (A and B) with station A located on the southern edge of the main channel 6,500 feet from the estuary's mouth and station B in center channel 47,500 feet from the mouth (16,500 feet from the head of the estuary).

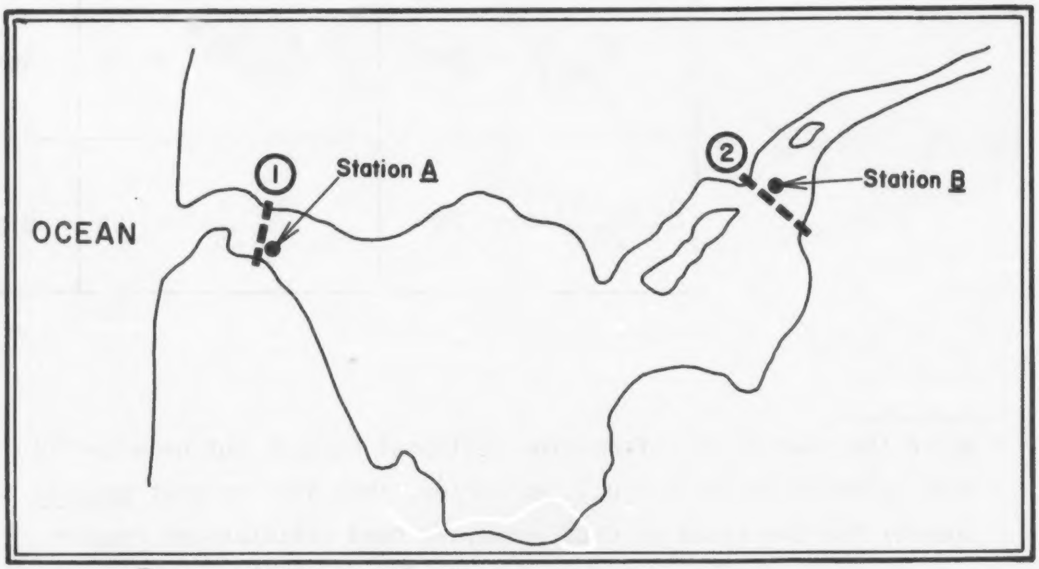


FIGURE VI-12 THE TYPICAL ESTUARY

Necessary salinity data were obtained from the coastal engineering department of the nearby university. USGS gage data were available for river flow, and, as a result of their own dredging program, the local district office of the U.S. Corps of Engineers could provide cross-sectional profiles in the approximate areas of both stations. The cross-sections are labeled (1) and (2) on Figure VI-12. The mean low tide depth reading on USGS Coastal Topographic maps was used to verify Corps data.* Current meters were borrowed from the School of Engineering at the nearby university and tied to buoy channel markers at A and B to provide velocity data not available through the coastal engineering department. The information obtained from these various sources is shown in graphical form in Figure VI-13.

The calculations proceeds as follows:

a. Stratification Parameter:

$$\frac{\Delta S}{S_o} = \frac{S_{bottom} - S_{surface}}{S_o} \rightarrow$$

STATION

A	B	
$\frac{33 - 30}{31.5} = \underline{.095}$	$\frac{14.5 - 10.5}{12.5} = \underline{.32}$	SUMMER
$\frac{31.5 - 24.2}{27.8} = \underline{.26}$	$\frac{4 - 2.1}{3.25} = \underline{.58}$	WINTER

---

* While the sources of information mentioned here do not necessarily have information for all U.S. estuaries, they are the best general sources for the kinds of data estuarine hand calculations require.

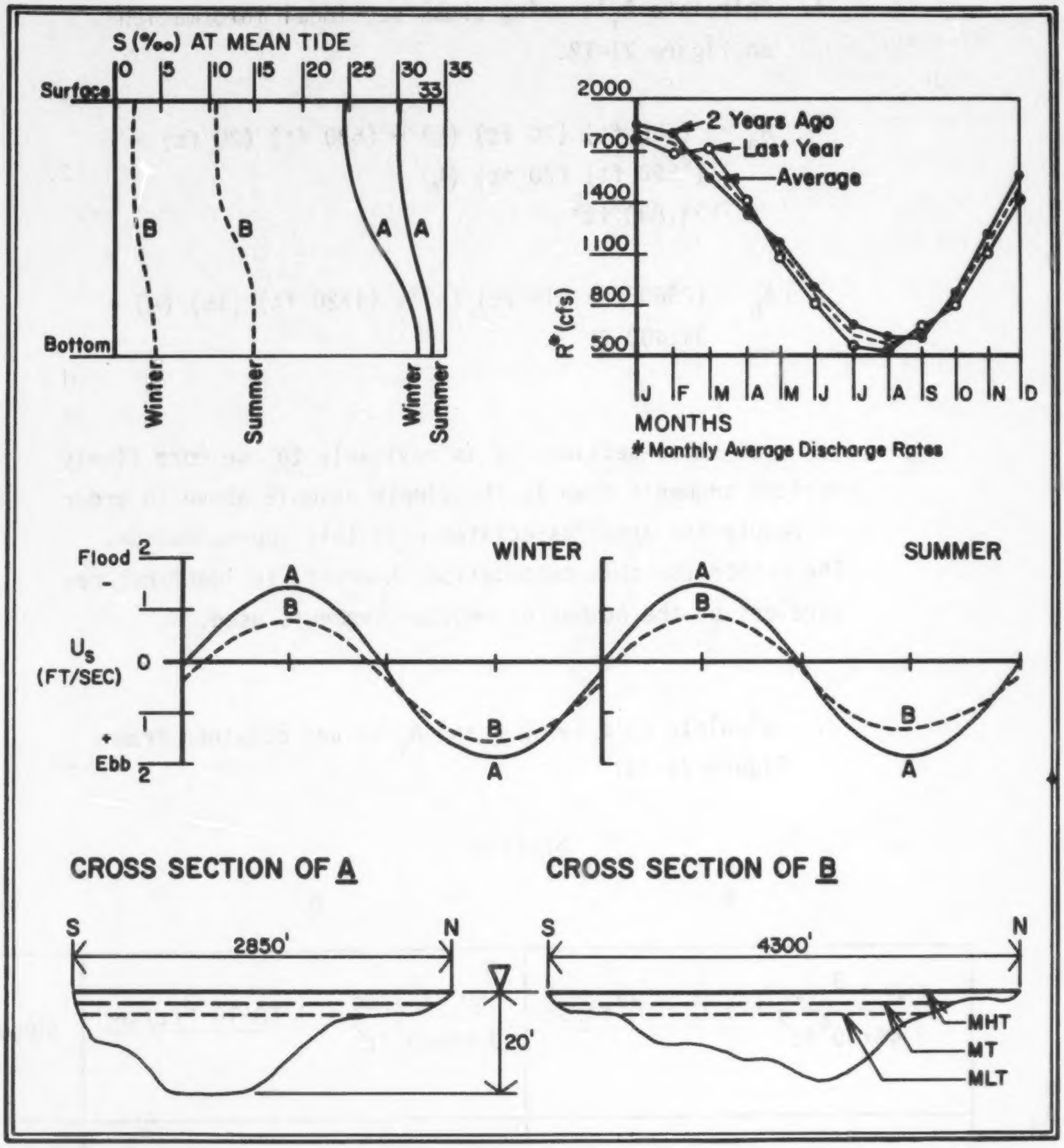


FIGURE VI-13 TYPICAL ESTUARY DATA FOR CLASSIFICATION CALCULATIONS

b. Circulation Parameter

1. Calculate A_i's using cross sectional information on figure VI-12.

$$A_a = (630\ \text{ft})\ (20\ \text{ft})\ (\tfrac{1}{2}) + (630\ \text{ft})\ (20\ \text{ft}) + (1590\ \text{ft})\ (20\ \text{ft})\ (\tfrac{1}{2}) = 34{,}800\ \text{ft}^2$$

$$A_b = (2580\ \text{ft})\ (16\ \text{ft})\ (\tfrac{1}{2}) + (1720\ \text{ft})\ (16)\ (\tfrac{1}{2}) = 34{,}400\ \text{ft}^2$$

For most cross-sections it is advisable to use more finely divided segments than in the simple example above in order to reduce the error associated with this approximation. The method for this calculation, however, is identical regardless of the number of regular segments used.

2. Calculate U_f's (with R and A_i values obtained from Figure VI-13)

STATION

	A	B	
$U_f = \frac{R}{A_i}$ →	$\frac{550\ \text{ft}^3/\text{sec}}{3.48\text{x}10^4\,\text{ft}^2} = 1.58\text{x}10^{-2}\,\text{ft/sec}$	$\frac{550\ \text{ft}^3/\text{sec}}{3.44\text{x}10^4\,\text{ft}^2} = 1.60\text{x}10^{-3}\,\text{ft/sec}$	SUMMER
	$\frac{1800\ \text{ft}^3/\text{sec}}{3.48\text{x}10^4\,\text{ft}^2} = 5.17\text{x}10^{-2}\,\text{ft/sec}$	$\frac{1800\ \text{ft}^3/\text{sec}}{3.44\text{x}10^4\,\text{ft}^2} = 5.23\text{x}10^{-2}\,\text{ft/sec}$	WINTER

3. Calculate $\frac{U_s}{U_f}$'s

U_s values are read from Figure VI-13. The precise value for U_s is the integral of the velocity curve (area under "ebb" velocity curve minus the area under the "flood" velocity curve) divided by the elapsed time period (length of one tidal cycle). If the elapsed time for flood flow at a station is only slightly below the elapsed time for ebb flow U_s may be approximated as $(U_{ebb(max)} - U_{Flood\ (Max)})/2$.

STATION

	A	B	
$\frac{U_s}{U_f}$ →	$\frac{0.15\ \text{ft/sec}}{1.58\text{x}10^{-2}\text{ft/sec}} = \underline{9.5}$	$\frac{0.3\ \text{ft/sec}}{1.60\text{x}10^{-2}\text{ft/sec}} = \underline{18.8}$	SUMMER
	$\frac{0.2\ \text{ft/sec}}{5.17\text{x}10^{-2}\text{ft/sec}} = \underline{3.9}$	$\frac{0.4\ \text{ft/sec}}{5.23\text{x}10^{-2}\text{ft/sec}} = \underline{7.65}$	WINTER

The circulation-stratification plots for Typical Estuary are shown in Figure VI-14 with points A_s (station A, summer value), A_w (station A, winter value), B_s (station B, summer value), and B_w (station B, winter values).

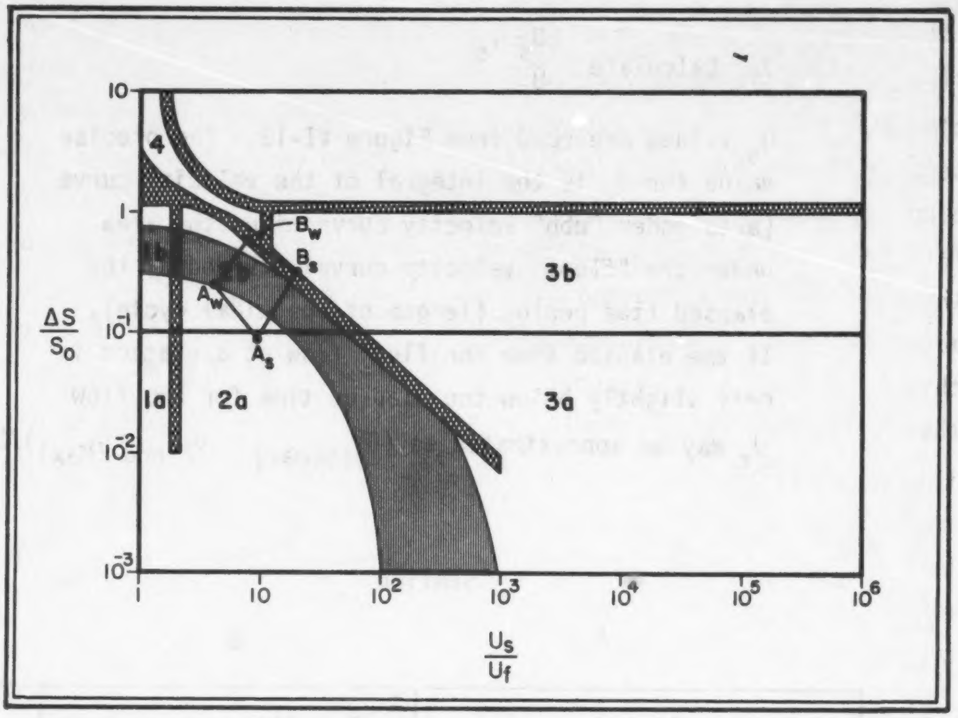


FIGURE VI-14 ESTUARINE CIRCULATION-STRATIFICATION DIAGRAM

As indicated, this estuary shows a significant amount of vertical stratification (especially at station B) but little evidence of major lateral non-homogeneity.

END OF EXAMPLE VI-1

6.3.4 Stratification-Circulation Diagram Interpretation

The closer an estuary falls to the lower left hand corner of a stratification-circulation diagram the more vertically and laterally homogenous it is, and the better that estuary fits the assumptions associated with this methodology. Thus the closer an estuary lies to that corner, the smaller the error that will be produced by the use of these hand calculations. On the stratification-circulation diagram

(Figure VI-9 or VI-14), two types of zonal demarcation can be seen. First are the diagonally striped divisions between adjacent estuarine classifications used by Hansen and Rattray to indicate a transitional period between separate classifications. The second is a wide solid band arching around the lower left corner of the diagram. Estuaries falling primarily inside of this band (lower left) are those for which the hand calculation method may be applied to obtain reasonably accurate results. Those estuaries falling predominately outside of this band violate the assumptions of this study to such an extent that hand calculations should not be used. If an estuary falls within this latter group, the planner should go directly to the use of computer models or other methods of water quality analysis. Within the band is a borderline or marginal zone. Hand calculations should be used for estuaries falling principally within this zone, however the accuracy of the calculations will be subject to increased uncertainty.

Turning to Figure VI-14, the Stratification-Circulation diagram for the Typical Estuary, it is apparent that this estuary lies principally within the marginal area. Moreover, the low flow classification (line A_s-B_s) also lies primarily within the marginal area. Thus, from Figure VI-7 the planner for the Typical Estuary should calculate additional criteria to help determine the suitability of using these water quality hand calculations. If the Typical Estuary plotted more predominately below the marginal zone, the planner could proceed with flushing time calculations since his estuary would then have met the classification criteria.

It should be noted that the data for the Typical Estuary produced a fairly tight cluster of data points. As can be seen in Figure VI-15 salinity profiles for one west coast estuary (the Alsea River and Estuary along the central Oregon coast) vary considerably more from season to season than did the Typical Estuary. This increased variation would produce a far greater spread in the summer and winter $\Delta S/S_o$ parameter values.

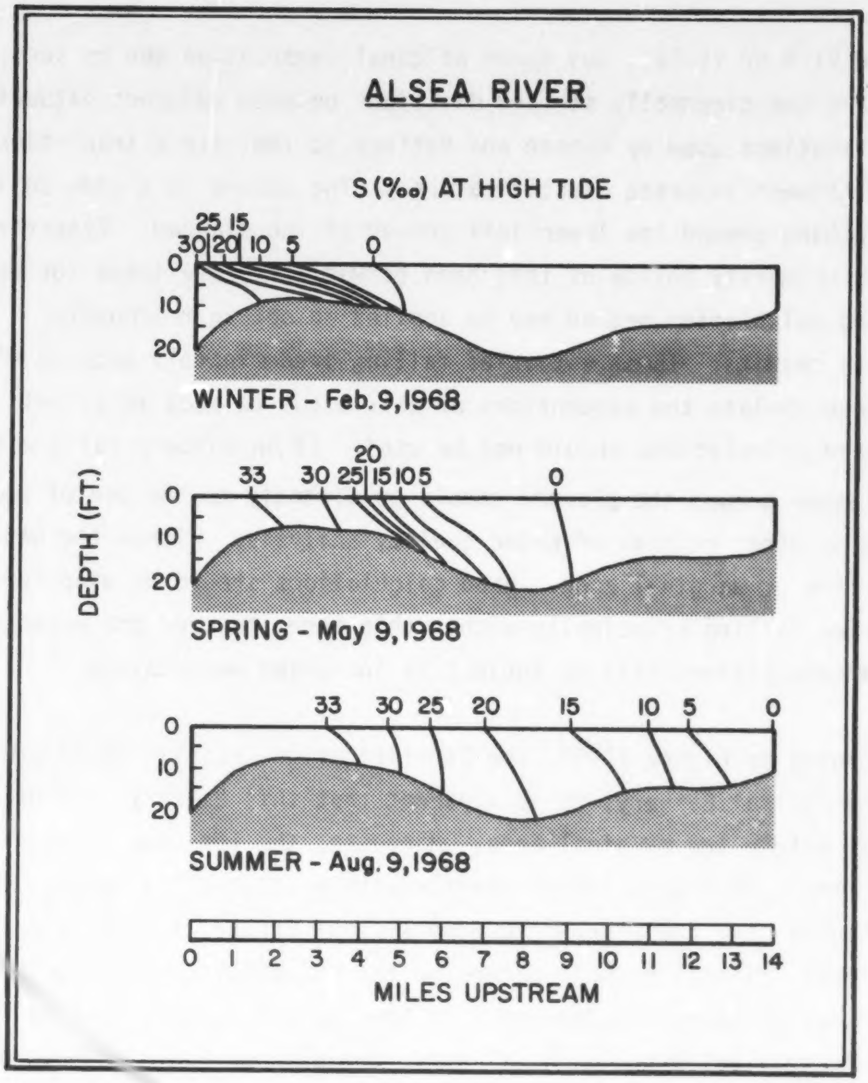


FIGURE VI-15 ALSEA ESTUARY SEASONAL, SALINITY VARIATIONS (FROM GIGER, 1972)

6.3.5 Flow Ratio Calculation

If application of the above classification procedure results in an ambiguous outcome regarding applicability of the hand calculation approach, another criterion should be applied. This is the flow ratio. Schultz and Simmons (1957) first observed the correlation between the flow ratio and estuary type. They defined the flow ratio for an estuary as:

$$FR = \frac{R}{P_t} \qquad \text{(VI-5)}$$

where

FR = the flow ratio,

R = the river flow measured over one tidal cycle (measured in ft^3)

and

P_t = the estuary tidal prism (in ft^3)

Thus the flow ratio compares the tidally induced flow in an estuary with the river induced flow. Schultz and Simmons observed that when this ratio was on the order of 1.0 or greater, the associated estuary was normally highly stratified. Conversely, ratios of about 0.1 or less were usually associated with very well-mixed estuaries and ratios in the range of 0.25 were associated with partially mixed estuaries. For the purposes of hand calculations, a flow ratio of 0.2 or less warrants inclusion of the estuary in the hand calculation process. Flow ratios in the range 0.2 to 0.3 should be considered marginal. Such estuaries may be included but only with the understanding that the hand calculations should produce increased error in water quality estimates. Finally, estuaries with flow ratios greater than 0.3 should not be included.

EXAMPLE VI-2

Flow Ratio Calculation

This example uses data for the Alsea Estuary in Oregon. River outflow is calculated for the low flow period of the year. Associated flow data were obtained from Oregon State University flow gages located approximately 2 miles above the estuary. From these data,

$$R = (104\ \text{ft}^3/\text{sec})\ (3600\ \text{sec/hour})\ (12.4\ \text{hrs/tidal cycle}) = 4.64 \times 10^6 \text{ft}^3/\text{tidal cycle}$$

Tidal prism data were available from previous work by Goodwin, *et al.*, (1970). From these data,

$$P_T = 5.05 \times 10^8 \text{ft}^3/\text{tidal cycle}$$

Thus

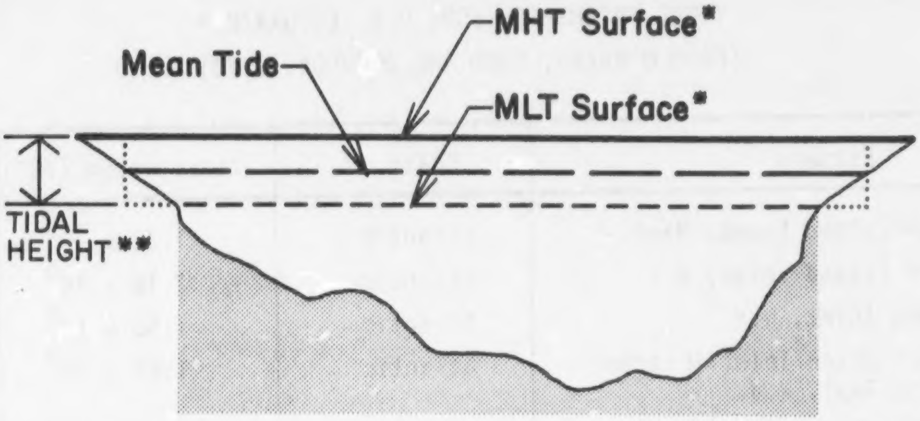
$$FR = \frac{R}{P_T} = \frac{4.64 \times 10^6}{5.05 \times 10^8} = 9.18 \times 10^{-3} < 0.1$$

Accordingly, Alsea Estuary should be considered well mixed in the summer low flow period, and would qualify for hand calculations.

END OF EXAMPLE VI-2

In Example VI-2, tidal prism data were available through a local university. This will be the case for many estuaries, both large and small, around the country. Where it is not the case, however, NOAA coastal charts may be used to estimate the difference between mean high tide and mean low tide estuary surface areas. As can be seen in the cross-section diagram in Figure VI-16 the estuarine tidal prism can be approximated by averaging the MLT and MHT surface areas and multiplying this averaged area by the local tidal height. Mean tidal heights (approximately 1 week before or after spring tides) should be used for this calculation. As indicated in Figure VI-16, the estuary can be conveniently subdivided into longitudinal sections for this averaging process, to reduce the resulting error. Table VI-1 lists tidal prisms estimated for many U.S. estuaries. These values may be used as an alternate to P_t calculations.

It should be emphasized that application of the flow ratio criterion is not a refinement of the stratification-circulation diagram method. In fact just the opposite is true. However, when the latter



P_T section = tidal height* $\frac{MHT + MLT}{2}$ width $\sim$ tidal height x MT width

P_T estuary = $\sum P_T$ for each section

* Widths obtained from or from MOAA tide table for the area

** Available from local Coast Guard Stations

FIGURE VI-16 ESTUARY CROSS-SECTION FOR TIDAL PRISM CALCULATIONS

method produces inconclusive results, use of the flow ratio which employs somewhat different estuarine characteristics can reduce the uncertainty of the outcome based on stratification-circulation data.

TABLE VI-1

TIDAL PRISMS FOR SOME U.S. ESTUARIES*

(FROM O'BRIEN, 1969 AND JOHNSON, 1973)

Estuary	Coast	Tidal Prism (ft^3)
Plum Island Sound, Mass.	Atlantic	1.32×10^9
Fire Island Inlet, N.Y.	Atlantic	2.18×10^9
Jones Inlet, N.Y.	Atlantic	1.50×10^9
Beach Haven Inlet (Little Egg Bay), N.J.	Atlantic	1.51×10^9
Little Egg Inlet (Great Bay), N.J.	Atlantic	1.72×10^9
Brigantine Inlet, N.J.	Atlantic	5.23×10^8
Absecon Inlet (before jetties), N.J.	Atlantic	1.65×10^9
Great Egg Harbor Entr, N.J.	Atlantic	2.00×10^9
Townsend Inlet, N.J.	Atlantic	5.56×10^8
Hereford Inlet, N.J.	Atlantic	1.19×10^9
Chincoteague Inlet, Va.	Atlantic	1.56×10^9
Oregon Inlet, N.C.	Atlantic	3.98×10^9
Ocracoke Inlet, N.C.	Atlantic	5.22×10^9
Drum Inlet, N.C.	Atlantic	5.82×10^8
Beaufort Inlet, N.C.	Atlantic	5.0×10^9
Carolina Beach Inlet, N.C.	Atlantic	5.25×10^8
Stono Inlet, S.C.	Atlantic	2.86×10^9
North Edisto River, S.C.	Atlantic	4.58×10^9
St. Helena Sound, S.C.	Atlantic	1.53×10^{10}
Port Royal Sound, S.C.	Atlantic	1.46×10^{10}
Calibogue Sound, S.C.	Atlantic	3.61×10^9
Wassaw Sound, Ga.	Atlantic	8.2×10^9
Ossabaw Sound, Ga.	Atlantic	6.81×10^9
Sapelo Sound, Ga.	Atlantic	7.36×10^9
St. Catherines Sound, Ga.	Atlantic	6.94×10^9

TABLE VI-1 (Cont.)

Estuary	Coast	Tidal Prism (ft^3)
Doboy Sound, Ga.	Atlantic	4.04×10^9
Altamaha Sound, Ga.	Atlantic	2.91×10^9
Hampton River, Ga.	Atlantic	1.01×10^9
St. Simon Sound, Ga.	Atlantic	6.54×10^9
St. Andrew Sound, Ga.	Atlantic	9.86×10^9
Ft. George Inlet, Fla.	Atlantic	3.11×10^8
Old St. Augustine Inlet, Fla.	Atlantic	1.31×10^9
Ponce de Leon, Fla. (before jetties)	Atlantic	5.74×10^8
Delaware Bay Entrance	Atlantic	1.25×10^{11}
Fire Island Inlet, N.Y.	Atlantic	1.86×10^9
East Rockaway Inlet, N.Y.	Atlantic	7.6×10^8
Rockaway Inlet, N.Y.	Atlantic	3.7×10^9
Masonboro Inlet, N.C.	Atlantic	8.55×10^8
St. Lucie Inlet, Fla.	Atlantic	5.94×10^8
Nantucket Inlet, Mass.	Atlantic	4.32×10^8
Shinnecock Inlet, N.Y.	Atlantic	2.19×10^8
Moriches Inlet, N.Y.	Atlantic	1.57×10^9 8.46×10^8
Shark River Inlet, N.J.	Atlantic	1.48×10^8
Manasguan Inlet, N.J.	Atlantic	1.75×10^8
Barnegat Inlet, N.J.	Atlantic	6.25×10^8
Absecon Inlet, N.J.	Atlantic	1.48×10^9
Cold Springs Harbor (Cape May), N.J.	Atlantic	6.50×10^8
Indian River Inlet, Del.	Atlantic	5.25×10^8
Winyah Bay, S.C.	Atlantic	3.02×10^9
Charleston, S.C.	Atlantic	5.75×10^9
Savannah River (Tybee Roads), Ga.	Atlantic	3.1×10^9

TABLE VI-1 (Cont.)

Estuary	Coast	Tidal Prism (ft^3)
St. Marys (Fernandina Harbor), Fla.	Atlantic	4.77×10^9
St. Johns River, Fla.	Atlantic	1.73×10^9
Fort Pierce Inlet, Fla.	Atlantic	5.81×10^8
Lake Worth Inlet, Fla.	Atlantic	9.0×10^8
Port Everglades, Fla.	Atlantic	3.0×10^8
Bakers Haulover, Fla.	Atlantic	3.6×10^8
Captiva Pass, Fla.	Gulf of Mexico	1.90×10^9
Boca Grande Pass, Fla.	Gulf of Mexico	1.26×10^{10}
Gasparilla Pass, Fla.	Gulf of Mexico	4.7×10^8
Stump Pass, Fla.	Gulf of Mexico	3.61×10^8
Midnight Pass, Fla.	Gulf of Mexico	2.61×10^8
Big Sarasota Pass, Fla.	Gulf of Mexico	7.6×10^8
New Pass, Fla.	Gulf of Mexico	4.00×10^8
Longboat Pass, Fla.	Gulf of Mexico	4.90×10^8
Sarasota Pass, Fla.	Gulf of Mexico	8.10×10^8
Pass-a-Grille	Gulf of Mexico	1.42×10^9
Johns Pass, Fla.	Gulf of Mexico	5.03×10^8
Little (Clearwater) Pass, Fla.	Gulf of Mexico	6.8×10^8
Big (Dunedin) Pass, Fla.	Gulf of Mexico	3.76×10^8
East (Destin) Pass, Fla.	Gulf of Mexico	1.62×10^9
Pensacola Bay Entr., Fla.	Gulf of Mexico	9.45×10^9
Perdido Pass, Ala.	Gulf of Mexico	5.84×10^8
Mobile Bay Entr., Ala.	Gulf of Mexico	2.0×10^{10}
Barataria Pass, La.	Gulf of Mexico	2.55×10^9
Caminada Pass, La.	Gulf of Mexico	6.34×10^8
Calcasieu Pass, La.	Gulf of Mexico	2.97×10^9
San Luis Pass, Tex.	Gulf of Mexico	5.84×10^8

TABLE VI-1 (Cont.)

Estuary	Coast	Tidal Prism (ft^3)
Venice Inlet, Fla.	Gulf of Mexico	8.5×10^7
Galveston Entr., Tex.	Gulf of Mexico	1.59×10^{10}
Aransas Pass, Tex.	Gulf of Mexico	1.76×10^9
Grays Harbor	Pacific	1.3×10^{10}
Willapa	Pacific	1.3×10^{10}
Columbia River	Pacific	2.9×10^{10}
Necanicum River	Pacific	4.4×10^7
Nehalem Bay	Pacific	4.3×10^8
Tillamook Bay	Pacific	2.5×10^9
Netarts Bay	Pacific	5.4×10^8
Sand Lake	Pacific	1.1×10^8
Nestucca River	Pacific	2.6×10^8
Salmon River	Pacific	4.3×10^7
Devils Lake	Pacific	1.1×10^8
Siletz Bay	Pacific	3.5×10^8
Yaquina Bay	Pacific	8.4×10^8
Alsea Estuary	Pacific	5.1×10^8
Siuslaw River	Pacific	2.8×10^8
Umpqua	Pacific	1.2×10^9
Coos Bay	Pacific	1.9×10^9
Caquille River	Pacific	1.3×10^8
Floras Lake	Pacific	6.8×10^7
Rogue River	Pacific	1.2×10^8
Chetco River	Pacific	2.9×10^7
Smith River	Pacific	9.5×10^7
Lake Earl	Pacific	5.1×10^8
Freshwater Lagoon	Pacific	4.7×10^7
Stove Lagoon	Pacific	1.2×10^8
Big Lagoon	Pacific	3.1×10^8

TABLE VI-1 (Cont.)

Estuary	Coast	Tidal Prism (ft^3)
Mad River	Pacific	2.4×10^7
Humbolt Bay	Pacific	2.4×10^9
Eel River	Pacific	3.1×10^8
Russian River	Pacific	6.3×10^7
Bodega Bay	Pacific	1.0×10^8
Tomales Bay	Pacific	1.0×10^9
Abbotts Lagoon	Pacific	3.5×10^7
Drakes Bay	Pacific	2.7×10^8
Bolinas Lagoon	Pacific	1.0×10^8
San Francisco Bay	Pacific	5.2×10^{10}
Santa Cruz Harbor	Pacific	4.3×10^6
Moss Landing	Pacific	9.4×10^7
Morro Bay	Pacific	8.7×10^7
Marina Del Rey	Pacific	6.9×10^7
Alamitos Bay	Pacific	6.9×10^7
Newport Bay	Pacific	2.1×10^8
Camp Pendleton	Pacific	1.1×10^7
Aqua Hedionda	Pacific	4.9×10^7
Mission Bay	Pacific	3.3×10^8
San Diego Bay	Pacific	1.8×10^9

*Includes some estuaries not necessarily appropriate for application of hand calculations.

6.4 FLUSHING TIME CALCULATIONS

6.4.1. General

The most basic water quality calculation which can be made for an estuary is the estuarine flushing time. The flushing time may be defined as the length of time required to replace existing fresh (river) water in an estuary at a rate equal to the river discharge. Thus, it represents the length of time (normally measured in tidal cycles) required to carry a pollutant out of the estuary (Dyer, 1973)*. Unless otherwise specified, estuarine flushing time is taken as the time required to flush a pollutant which was introduced into the estuary at its head (i.e. with the river inflow).

As has been mentioned, the net non-tidal flow in an estuary is always seaward** and is dependent on the river discharge. This nontidal flow is the driving force behind estuarine flushing. Were it not for this advective displacement of pollutants only dispersion and diffusion forces would dilute pollutant concentration within the oscillatory motion of the tides. Tomales Bay, California, can be used as an example to show the importance of this river induced net motion. This small, elongated California bay has essentially no fresh water inflow. As a result there is no advective seaward motion (and pollutant removal is dependent upon dispersion and diffusion processes). The flushing time for the bay is approximately 140 days (Johnson, et al., 1961). This can be compared with Alsea Estuary in Oregon having a flushing time of approximately 8 days (see example later in this section), with the much larger St. Croix Estuary in Nova Scotia having a flushing time of approximately 8 days (Ketchum and Keen, 1951), or with the very large Hudson River Estuary with a low flow flushing time of approximately 10.5 days (Ketchum, 1950).

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*Actually this length of time will not remove all of a dispersed pollutant from an estuary. Some portion will remain behind. However it is sufficiently small so that it can be ignored without creating a significant error.

** While net flow is always seaward for the estuaries being considered here, it is possible to have a net upstream flow in individual embayments of an estuary. While this occurance is very rare in the United States, an example of such a situation is the South Bay of San Francisco Bay where fresh water inflows are so small that surface evaporation exceeds inflow. Thus, net flow is upstream during most of the year.

6.4.2 Procedure

Flushing times for a given estuary will vary over the course of a year as river discharge varies. The critical time is the low river flow period since this period corresponds with the slowest flushing rates. The planner may also want to calculate the best flushing characteristics (high river flow) for an estuary. In addition to providing a more complete picture of the estuarine system, knowledge of the full range of annual flushing variations can be useful in evaluating the impact of seasonal discharges (e.g. fall and winter cannery operation in an estuary with a characteristic summer fresh water low flow). Further, storm sewer runoff will more normally coincide with these best flushing conditions (high flow) and not with the low flow, or poorest flushing characteristic. Thus analysis of storm runoff is often better suited for high flow flushing conditions. However, the low flow calculation should be used for primary planning purposes.

Several methods are available for approximate calculation of flushing time (Dyer, 1973). Two will be presented here, and while data requirements for both are minimal, one (fraction of fresh water method) will normally overestimate flushing time, and the other (modified tidal prism method) will normally slightly underestimate flushing time. The limitations and advantages of each are listed below.

6.4.3 Fraction of Fresh Water Method

This method uses the mean salinity level in a segment of the estuary. The fraction of fresh water within that segment is calculated by comparing segment mean salinity to ocean water salinity. The resulting volume of fresh water is then compared to the river discharge rate to calculate the length of time required to replace all the freshwater within that estuarine segment.

The principal limitation of this method is that it assumes uniform salinity levels throughout the segment. A second limitation is that it assumes that during each tidal cycle a volume of water equal to the river discharge moves into a given estuarine segment from the adjacent upstream segment, and that an equal volume of the water originally in the segment moves on to the adjacent one downstream. Once this exchange has taken place, the water within each segment is assumed to be instantaneously and completely mixed and to again become a homogeneous water mass. Proper selection of estuarine segments can reduce the error created by the first of these limitations. This error will also be reduced by the use of low flow data since estuaries tend to be closer to well mixed during low river discharge periods. The second limitation described above is inherent in the methodology and cannot be counteracted. Normally, however, the error it introduces is acceptably small.

A primary advantage of this method is that the derived fraction of fresh water coefficients will form the basis for later segment dispersion coefficient calculations. Secondarily, this method is quite accurate when used on truly well-mixed estuaries.

6.4.4 Modified Tidal Prism Method

This method divides an estuary into segments whose lengths are defined by the maximum excursion path of a water particle during a tidal cycle. Within each segment the tidal prism is compared to the total segment volume as a measure of the flushing potential of that segment per tidal cycle (Dyer, 1973). The method assumes complete mixing of the incoming tidal prism waters with the low water volumes within each segment. Best results have been obtained in estuaries when the number of segments is large (i.e. when river flow is very low) and when estuarine cross-sectional area increases fairly quickly downstream (Dyer, 1973).

The modified tidal prism method has the advantage of not requiring knowledge of the salinity distribution and also will provide some concept of mean segment velocities since each segment length is tied to particle excursion length over one tidal cycle.

6.4.5 Calculation Methodology

6.4.5.1 Fraction of Fresh Water Method

This is a six step procedure. These steps are:

1) Graph the estuarine salinity profiles.
2) Divide the estuary into segments. There is no minimum or maximum number of segments required, nor should each segment be of the same length. The divisions should be selected so that mean segment salinity is relatively constant over the full length of the segment. Thus, stretches of steep salinity gradient will have short segments and stretches where salinity remains constant may have very long segments. Example VI-3 provides an illustration.

3) Calculate each segment's fraction of fresh water by:

$$f_i = \frac{S_s - S_i}{S_s} \qquad \text{(VI-6)}$$

where f_i = fraction of fresh water for segment "i"

S_s = salinity of sea water* (∿ 34 ^{o}/oo)

and S_i = mean salinity for segment "i"

4) Calculate the quantity of fresh water in each segment by:

$$W_i = f_i \times vol_i \qquad \text{(VI-7)}$$

where W_i = quantity of fresh water in segment "i"
and vol_i = low tide volume of segment "i"

5) Calculate the exchange time (flushing time) for each segment by:

$$T_i = W_i / R \qquad \text{(VI-8a)}$$

where T_i = **segment** flushing time

and R = river discharge over one tidal cycle and 6) Calculate the entire estuary flushing time by

---

*Sea surface salinity along US shores vary spatially. Neuman and Pierson (1966) mapped Pacific mean coastal surface salinities as varying from 32.4 ^{o}/oo at Puget Sound to 33.9 ^{o}/oo at the US - Mexico border; Atlantic mean coastal surface salinities as varying from 32.5 ^{o}/oo in Maine to 36.2 ^{o}/oo at the southern extreme of Florida; and Gulf coast salinities as varying between 36.2 ^{o}/oo and 36.4 ^{o}/oo. Surface coastal salinities in Long Island Sound (Hardy, 1972) and off Long Island south coast (Hydroscience, 1974) vary between 26.5 and 28.5 ^{o}/oo.

$$T_f = \sum_{i=1}^{n} T_i \tag{VI-8b}$$

where T_f = estuary flushing time

n = number of segments.

EXAMPLE VI-3

Flushing Time Calculation By Fraction Of Fresh Water Method

This example pertains to Alsea Estuary in Oregon. Necessary estuarine topographical data were obtained from a report by Goodwin, Emmett and Glenne (1970). Salinity data were obtained from that report as well as from a report by McKinsey (1974). Sea surface salinity was estimated to be 34 $^{0}/oo$. The profile of high tide salinity levels along the estuary is shown in Figure VI-17. The selected segmentation scheme is shown in Figure VI-18 with mean salinity levels. Where the salinity increases fairly regularly along the length of the estuary the dividing lines between segments is somewhat arbitrary. Where changes in mean salinity occur sharply, the location of these segment divisions is more fixed. Data showing cross sectional areas were obtained as shown from Goodwin, Emmett and Glenne (1970) (Figure VI-19), and segment divisions are also shown on that figure. The data presented in the three Figures VI-17, 18, and 19 are summarized in Table VI-2 for use in flushing time calculations.

This effort completes steps 1 and 2 of the 6 step procedure.

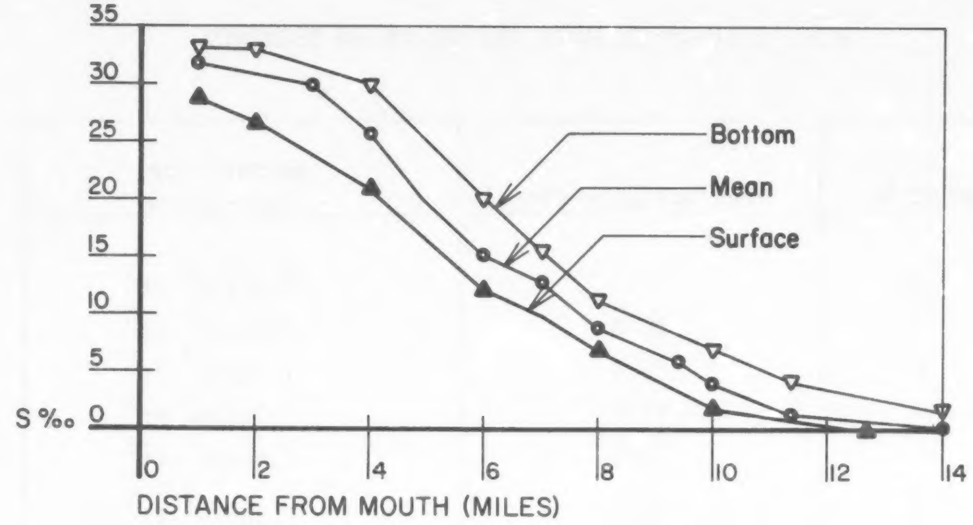

FIGURE VI-17 ALSEA ESTUARY SUMMER HIGH TIDE SALINITY PROFILE

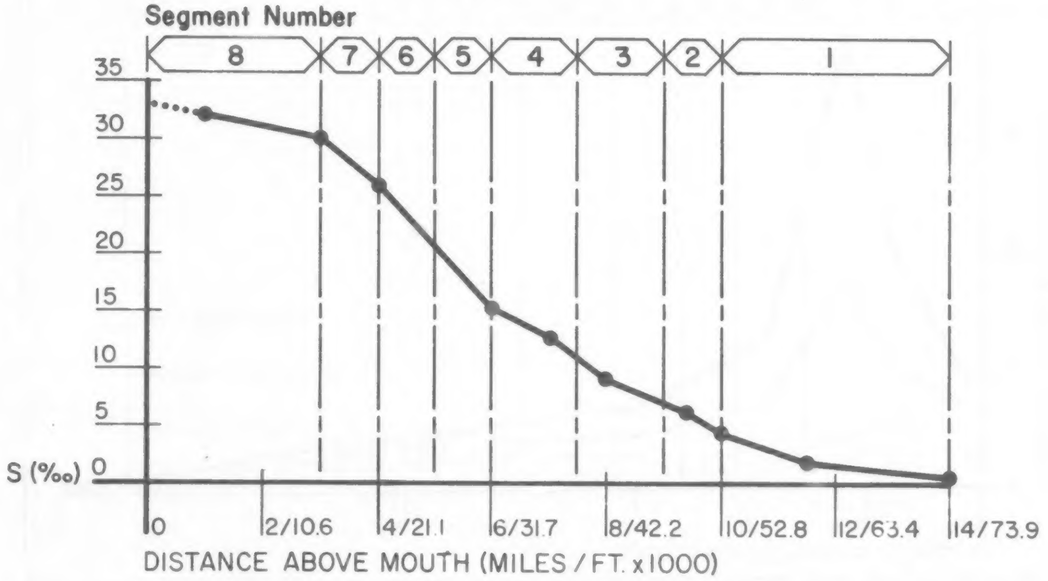

FIGURE VI-18 ALSEA ESTUARY FRACTION OF FRESH WATER SEGMENTATION

TABLE VI-2

ALSEA ESTUARY SALINITY AND VOLUME BY SEGMENTS

Segment No.	Mean Salinity (‰)	Segment Low Tide-Volumn
8	31.3	7.92×10^7 ft^3
7	28.0	1.28×10^7 ft^3
6	23.3	1.06×10^7 ft^3
5	17.7	1.37×10^7 ft^3
4	12.3	2.45×10^7 ft^3
3	8.1	2.30×10^7 ft^3
2	4.9	1.32×10^7 ft^3
1	1.8	3.59×10^7 ft^3

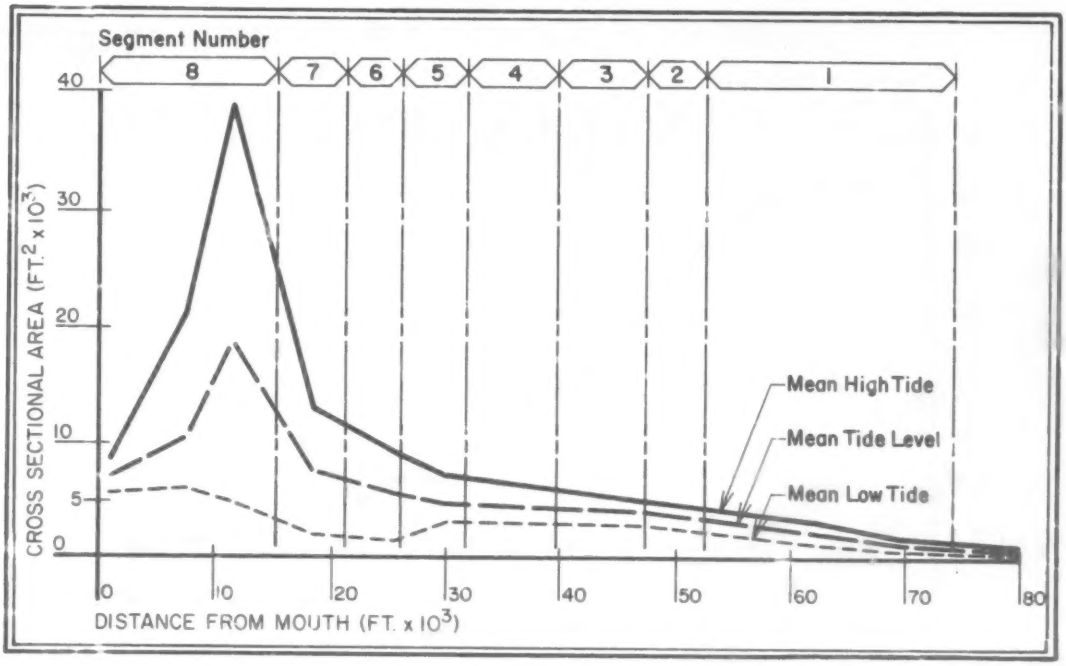

FIGURE VI-19 ALSEA ESTUARY CROSS SECTIONAL AREAS

3) Calculate f_i's by:

$$f_i = \frac{S_s - S_i}{S_i}$$

Segment	S_i (o/oo)	f_i
8	31.3	0.0794
7	28.0	0.176
6	23.3	0.315
5	17.7	0.479
4	12.3	0.638
3	8.1	0.762
2	4.9	0.856
1	1.8	0.947

4) Calculate W_i's by:

$$W_i = f_i \times Vol_i$$

Segment	f_i	Vol_i ($x10^7$ ft^3)	Q_i ($x10^7$ ft^3)
8	0.0794	7.92	0.629
7	0.176	1.28	0.225
6	0.315	1.06	0.334
5	0.479	1.37	0.656
4	0.638	2.45	1.56
3	0.762	2.30	1.75
2	0.856	1.32	1.13
1	0.947	3.59	3.40

5) Calculate T_i by:

$$T_i = \frac{W_i}{R}$$

When $R = (104\ ft^3/sec)\ (3600\ sec/hr)\ (12.4\ hr/tidal\ cycle)$

$R = 4.64x10^6\ ft^3/tidal\ cycle$ (from McKinsey, 1974)

Segment	Q_i ($x10^6$ ft^3)	T_i
8	6.29	1.36
7	2.25	0.485
6	3.34	0.720
5	6.56	1.41
4	15.6	3.36
3	17.5	3.77
2	11.3	2.41
1	34.0	7.33

6) Sum T_i's to obtain T_f by:

$$T_f = \sum_{i=1}^{8} T_i$$

$$T_f = (1.36 + 0.485 + 0.72 + 1.41 + 3.36 + 3.77 + 2.41 + 7.33)$$

$$\underline{T_f = 20.8\ tidal\ cycles}$$

$$\underline{or\ T_f = 10.7\ days}$$

END OF EXAMPLE VI-3

In addition to producing a total estuarine flushing time this method provides some feeling for the way that pollutants move through an estuary. Note that the total flushing time for segments 6, 7, and 8 is approximately 2.5 tidal cycles while the distance covered by the pollutant is 5 miles. Conversely, the 8 miles at the head of Alsea Estuary require approximately 16.9 tidal cycles to flush a pollutant. This indicates that once a pollutant gets near the mouth of Alsea Estuary it is quickly flushed out of the estuary. However, pollutants introduced near the estuary's head require a considerable length of time to move downstream into the main body of the estuary. The flushing capacity of an estuary is much better at the mouth of the estuary than at the head. While this is typical of all estuaries the difference in flushing rate from mouth to head is not always as large as in Alsea Estuary.

6.4.5.2 Modified Tidal Prism Method

This is a four step methodology. The steps are:

1) Segment the estuary. For this method an estuary must be segmented so that each segment length reflects the excursion distance a particle can travel during one tidal cycle. The innermost section must then have an intertidal volume completely supplied by river flow. Thus,

$$P_0 = R \qquad \text{(from Dyer, 1973)}$$

Where P_0 = Tidal prism (intertidal volume) of segment "0"

and R = River discharge over one tidal cycle

The low tide volume in this section (V_0) is that water volume below low tide occupying the space under the intertidal volume P_0 (which has just been defined as being equal to R). The seaward limit of the next seaward segment is placed such that its low tide volume (V_1) is defined by:

$$V_1 = P_0 + V_0$$

P_1 is then that intertidal volume which, at high tide, resides above V_1. Successive segments are defined in an identical manner to this segment so that:

$$V_i = P_{i-1} + V_{i-1} \qquad \text{(VI-9)}$$

Thus each segment contains, at high tide, the volume of water contained in the next seaward section at low tide.

2) Calculate the exchange ratio (r) by:

$$r_i = \frac{P_i}{P_i + V_i} \qquad \text{(VI-10)}$$

Thus the exchange ratio for a segment is a measure of a portion of water associated with that segment which is exchanged with adjacent segments during each tidal cycle.

3) Calculate segment flushing time by:

$$T_i = {}^1/_{r_i} \qquad \text{(VI-11)}$$

Where T_i = Flushing time for segment "i"

4) Calculate total estuarine flushing time by:

$$T_f = \sum_{i=1}^{n} T_i$$

Where T_f = Total estuary flushing time

and n = Number of segments.

EXAMPLE VI-4

Estuary Flushing Time Calculation By Modified Tidal Prism Method

1) Estuary Segmentation: Again, the Alsea Estuary will be used as an example. From previous calculations, $R = 4.64\text{x}10^6\ \text{ft}^3$/tidal cycle.

Thus $P_o = R = 4.64\text{x}10^6\ \text{ft}^3$

The differences in area between high tide and low tide cross sectional areas on Figure VI-20 (a reproduction of the basic cross sectional information in Figure VI-19) is the intertidal area, or tidal prism. Thus, from the head of the estuary you can measure the equivalent of "R" in intertidal volume (intertidal cross sectional area times segment length). Alternately, this intertidal volume can be graphed as in Figure VI-21. In this figure the intertidal volume upstream of any point is plotted against the distance from the mouth of Alsea Estuary. Also shown on Figure VI-21 is a horizontal line from R ($4.64 \times 10^6\ \text{ft}^3$) to Alsea Estuary intertidal volume and down to the distance from the mouth of Alsea Estuary above which $P_o = R$. V_o may now be calculated as the low tide volume above 68,500 ft from the estuary's mouth. From Figure VI-20, $V_o = 4.85 \times 10^6\ \text{ft}^3$ and the length of segment "0" is 74,000 ft* - 68,500 ft = 5,500 ft.

Then,

$$V_1 = P_o + V_o$$

$$V_1 = 4.64\text{x}10^6\ \text{ft}^3 + 4.85\text{x}10^6\ \text{ft}^3$$

$$V_1 = 9.49\text{x}10^6\ \text{ft}^3$$

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* The extent of mean summer salinity intrusion in Alsea estuary (i.e., it's length) is approximately 74,000 ft.

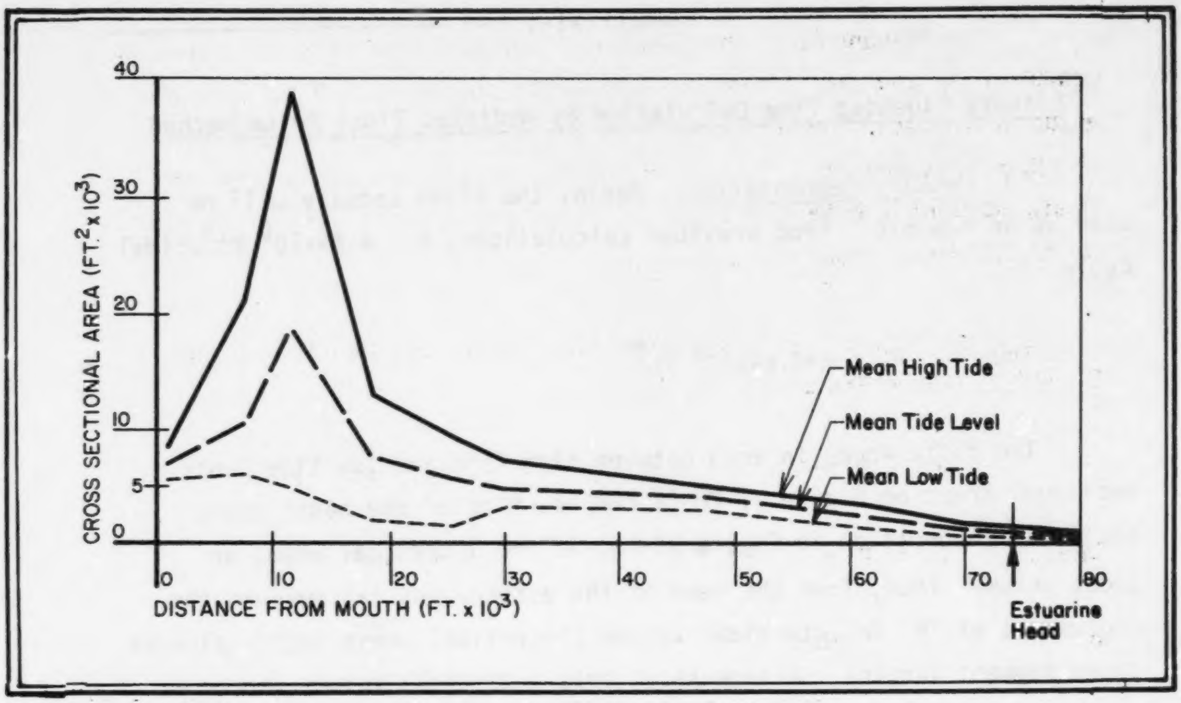

FIGURE VI-20 ALSEA ESTUARY CROSS-SECTIONAL AREAS

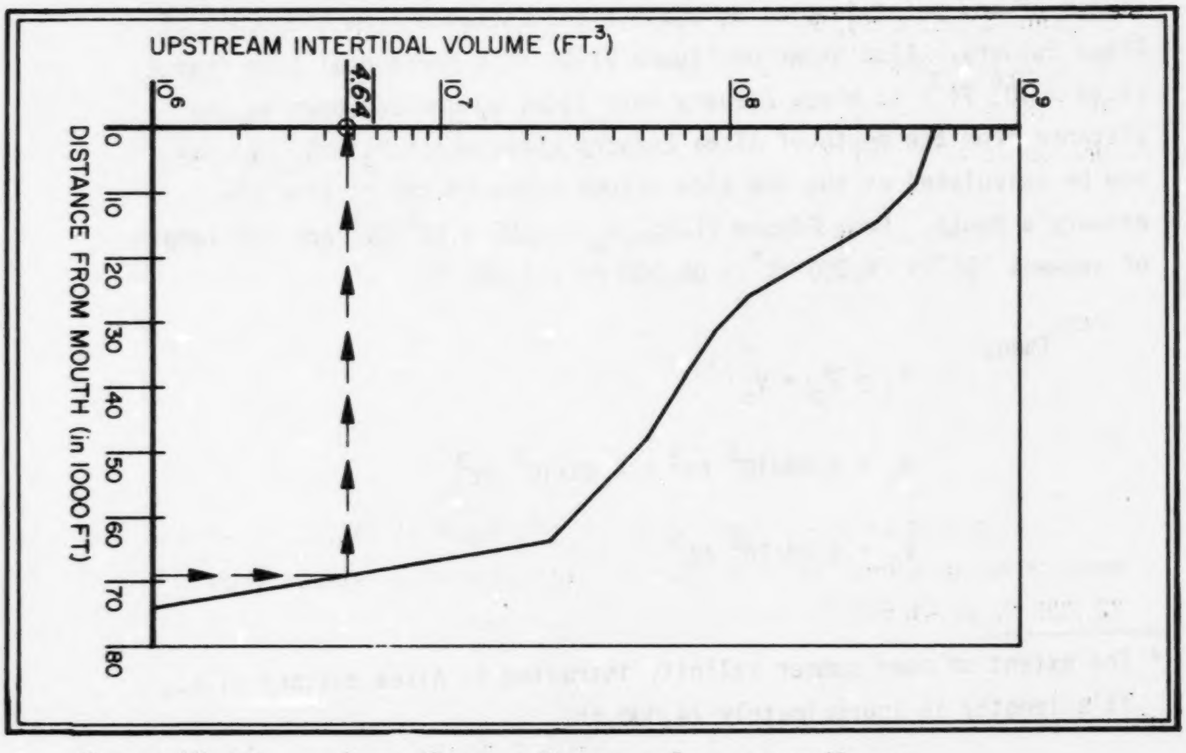

FIGURE VI-21 ALSEA ESTUARY UPSTREAM INTERTIDAL VOLUME

From Figure VI-20 the length of segment 1 may be calculated by measuring a low tide volume = $9.49\text{x}10^6$ ft^3 below 68,500 ft from the estuary's mouth. From Figure VI-20 the distance below 68,500 ft which corresponds to a $V_1 = 9.49\text{x}10^6\ \text{ft}^3$ is approximately 8250 ft. Thus, segment 1 extends from 60,250 feet above Alsea estuary's mouth to 68,500 feet above the estuary.

From Figure VI-21 the intertidal volume between 60,250 and 68,500 ft above the estuarine mouth is:

$$P_1 = 2.75\text{x}10^7\ \text{ft}^3 - 4.64\text{x}10^6\ \text{ft}^3 = 2.286\text{x}10^7\ \text{ft}^3$$

Then, $$V_2 = P_1 + V_1 = 2.29\text{x}10^7\ \text{ft}^3 + 9.49\text{x}10^6\ \text{ft}^3$$

or $$V_2 = 3.24\text{x}10^7\ \text{ft}^3$$

Following this same procedure, from Figure VI-20 the length of segment 2 below 60,250 ft from the estuary's mouth which corresponds to a $V_2 = 3.24\text{x}10^7\ \text{ft}^3$ is 15,700 ft with a mean segment cross section of 2050 ft^2. Thus, segment 2 extends from 44,550 ft to 60,250 ft above the mouth of Alsea Estuary. From Figure VI-21 this corresponds to:

$$P_2 = 5.4\text{x}10^7\ \text{ft}^3 - 2.75\text{x}10^7\ \text{ft}^3 = 2.65\text{x}10^7\ \text{ft}^3$$

Then,

$$V_3 = P_2 + V_2 = 2.65\text{x}10^7\ \text{ft}^3 + 3.24\text{x}10^7\ \text{ft}^3$$

or $$V_3 = 5.89\text{x}10^7\ \text{ft}^3$$

From Figure VI-20 the length of segment 3 is 22,250 ft with a mean cross sectional area of 2650 ft^2 and segment 3 extends from 22,300 ft to 44,550 ft above the estuary mouth.

From Figure VI-21

$$P_3 = 1.7\text{x}10^8 \text{ ft}^3 - 5.4\text{x}10^7 \text{ ft}^3 = 1.16\text{x}10^8 \text{ ft}^3$$

Thus $$V_4 = P_3 + V_3 = 1.16\text{x}10^8 \text{ ft}^3 + 5.89\text{x}10^7 \text{ ft}^3$$

$$V_4 = 1.74\text{x}10^8 \text{ ft}^3$$

However, this V_4 is greater than the remaining low tide volume in Alsea Estuary. Thus, segment 4 extends from the mouth to 22,300 ft above the mouth with a segment length of 22,300 ft, a mean cross sectional low tide area of 3825 ft^2 and,

$$V_4 = 3825 \text{ ft}^2\text{x}22{,}300 \text{ ft} = 8.525\text{x}10^7 \text{ ft}^3$$

From Figure VI-21 the intertidal volume below 22,300 ft from the estuary's mouth is:

$$P_4 = 5.1\text{x}10^8 - 1.7\text{x}10^8 = 3.4\text{x}10^8 \text{ ft}^3$$

Table VI-3 summarizes these segment physical characteristics.

TABLE VI-3

ALSEA ESTUARY SEGMENTATION FOR MODIFIED TIDAL PRISM METHOD

Segment	length (ft)	V_i (ft^3)	P_i (ft^3)
0	5,500	$4.85\text{x}10^6$	$4.64\text{x}10^6$
1	8,250	$9.49\text{x}10^6$	$2.286\text{x}10^7$
2	15,700	$3.24\text{x}10^7$	$2.65\text{x}10^7$
3	22,250	$5.89\text{x}10^7$	$1.16\text{x}10^8$
4	22,300	$8.525\text{x}10^7$	$3.4\text{x}10^8$

2) Calculate r_i's by:

$$r_i = \frac{P_i}{V_i+P_i}$$

Segment	P_i (10^6 ft^3)	V_i(10^6x ft^3)	r_i
0	4.64	4.85	0.489
1	22.86	9.49	0.707
2	26.5	32.4	0.450
3	116.0	58.9	0.663
4	340.0	85.25	0.800

3) Calculate T_i's by:

$$T_i = \frac{1}{r_i}$$

Segment	r_i	T_i
0	0.489	2.25
1	0.707	1.62
2	0.450	2.32
3	0.663	1.61
4	0.800	1.25

4) Calculate T_f:

$$T_f = \sum_{i=o}^{4} T_i$$

$$T_f = 2.25 + 1.62 + 2.32 + 1.61 + 1.25$$

$$T_f = 9.05 \text{ tidal cycles}$$

or $$T_f = 4.7 \text{ days}$$

END OF EXAMPLE VI-4

This flushing time estimate for Alsea estuary is less than 50% of that by the fraction of fresh water method. The discrepancy is unusually large and should be smaller for most U.S. estuaries. As was stated earlier the modified tidal prism method produces best results when the number of segments is large which is not the case for this estuary.* A planner, of course, has no control over the number of segments created in a specific estuary with a specific river outflow. However, the larger the estuary with respect to river outflow, the greater the number of segments and the greater the accuracy of the flushing calculations. Based on these two flushing calculations the planner for Alsea Estuary could estimate the flushing time for that estuary to be about 8 days and could be reasonably assured that the actual instantaneous flushing time would not vary significantly from this figure without major changes in river discharge.

In addition to estimating pollutant residance times, the flushing time for an estuary, or more particularly, for a segment of an estuary, has a direct relationship with the concentration of a pollutant in that segment. Segments with high flushing times will concentrate pollutants and will thus tend to be the sites of the most severe water quality problems. This relationship will be explained in considerably more detail in the next section of this chapter.

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* Ketchum (1955) obtained 13 segments in his analysis of the Raritan River and got good agreement between predicted and observed values.

Both methods presented here for estimation of segment and estuarine flushing times are somewhat empirical and are subject to considerable error. However, the great utility of the flushing time and its relative ease of calculation make it worth the effort. These simple procedures for estimation of segment flushing times provides: pollutant residence times, locations of peak pollutant storage concentration, segment flushing time and estuarine flushing time. This information is extremely valuable for both subsequent calculation of pollutant distribution and for gaining a basic understanding of the kinetic operation of an estuary.

6.5 CALCULATION OF POLLUTANT CONCENTRATIONS IN ESTUARIES

6.5.1 General

The previous two major sections were preliminary sections to this section. By classifying each estuary and by calculating segment and estuarine flushing time, the basic ground work has been completed to now directly address estuarine water quality. The methods presented in this section will allow direct calculation of the concentration of a wide range of pollutants in the estuarine environment which will act as primary water quality indicators (e.g., heavy metals, BOD, hydrocarbons, coliform bacteria, pesticides, oil and grease, and nutrients*). Input waste loadings can be divided into four cases which will be addressed separately: continuous flow conservative discharges, continuous flow non-conservative discharges, single event or one-time conservative discharges, and one-time non-conservative discharges. Actually, a continuous discharge will probably contain both conservative and nonconservative substances and thus an analysis of both cases will be required for a full evaluation of its impact on estuarine water quality.

---

*Nutrients will be treated in this section as a conservative pollutant. While it is true that nutrients are consumed for phytoplankton production and are thus not fully conserved, light, and not nutrients, is normally the limiting factor for estuarine production. Therefore, the assumption that nutrients will be conservative will not introduce excessive error.

This methodology will address only one discharge at a time and is thus designed primarily for use in assessing changes in water quality resulting from changes (additions or deletions) in the waste loading pattern. This type of analysis requires only that a few discharges be evaluated and that the results of pollution dispersion be added to the existing, or background, pollutant concentrations. The approach can also be used to assess existing water quality. However, this requires separate analysis of each waste load and generally will be quite time consuming.

A simple schematic representation of individual steps is shown in Figure VI-22. The types of discharges associated with each step in this section are listed in Figure VI-22 as are water quality parameters appropriate for the analysis in each of these steps.

As was true for estuarine hydrodynamics, pollutant dispersion in the estuarine environment is a very complex process. In fact, this is not a single process, but is the composite of a number of individual physical and chemical processes including advective dispersion, eddy diffusion, and density gradient diffusion. The concern here is to approximate only the cumulative effect of all of these separate mixing processes. The methods presented in this section are designed to approximate this steady state distribution of pollutants based on known flushing times and on estimated decay rates for non-conservative pollutants. Finally, single event discharges (non-steady state events) will be estimated through the use of dispersion equation techniques.

6.5.2 Continuous Flow Conservative Pollutants

The concentration of a conservative pollutant entering an estuary in a continuous flow will vary as a function of the location of the entry point location. It will be convenient to separate pollutants entering an estuary at the head of the estuary (with the river discharge)

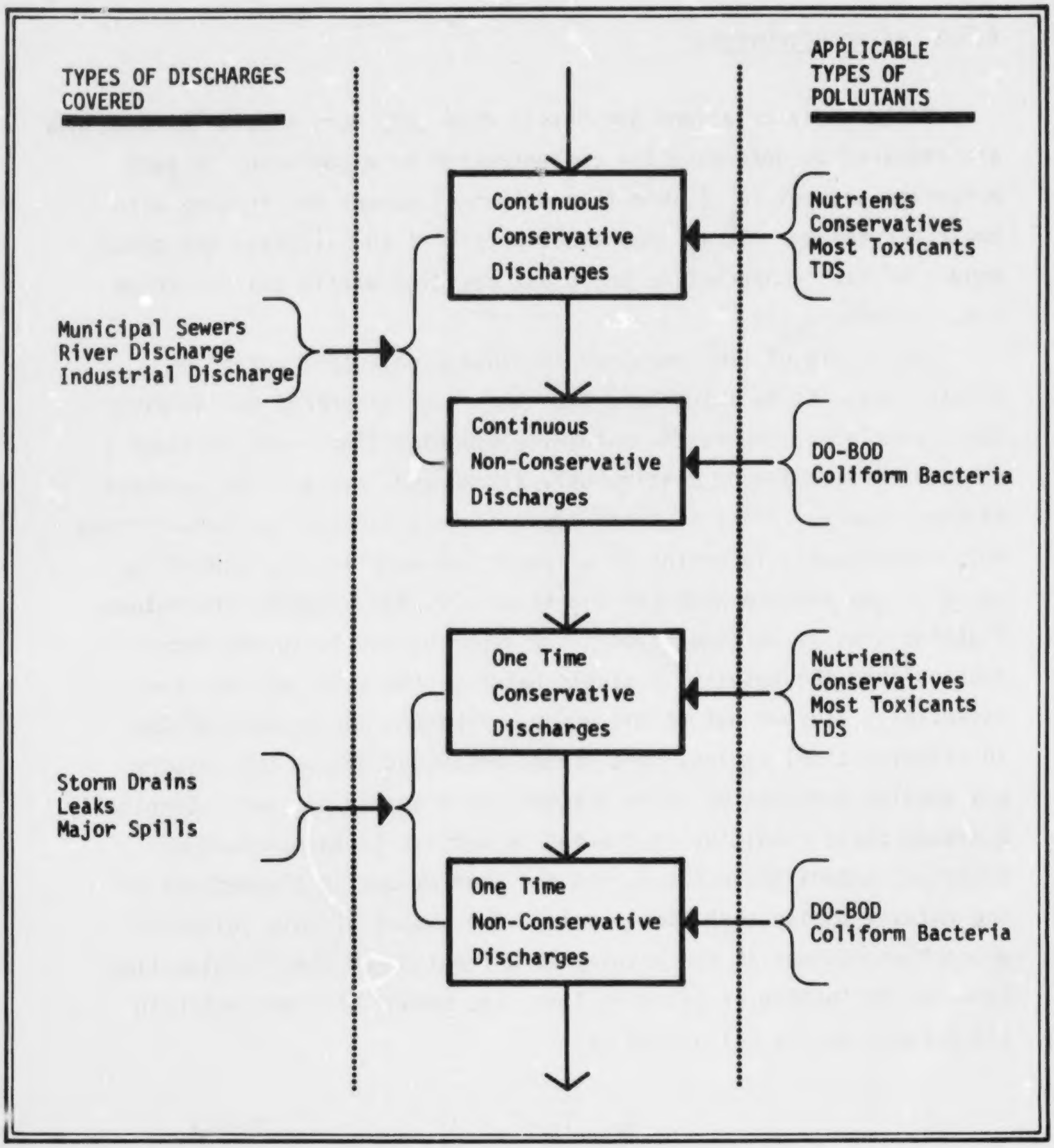
TYPES OF DISCHARGES COVERED
APPLICABLE TYPES OF POLLUTANTS
Continuous Conservative Discharges
Nutrients
Conservatives
Most Toxicants
TDS
Municipal Sewers
River Discharge
Industrial Discharge
Continuous Non-Conservative Discharges
DO-BOD
Coliform Bacteria
One Time Conservative Discharges
Nutrients
Conservatives
Most Toxicants
TDS
Storm Drains
Leaks
Major Spills
One Time Non-Conservative Discharges
DO-BOD
Coliform Bacteria

FIGURE VI-22 DISPERSION METHODOLOGY SCHEMATIC

from those entering along the estuary's sides. The two impacts will then be addressed separately.

6.5.3 River Discharges

Based on calculations previously done only very simple calculations are required to determine the concentration of a pollutant in each estuarine segment (c_{pi}) when that pollutant enters the estuary with the river inflow. It is then equally trivial to calculate the total amount of each conservative pollutant residing within the estuarine water column (W_{pE}).

The length of time required to flush a pollutant out of an estuary after it is introduced with the river discharge has already been calculated, and is the estuarine flushing time. Now consider a conservative pollutant continuously discharged into a river upstream of the estuary. After a single tidal cycle's worth of pollutant flows into the estuary, it begins to disperse and work its way toward the mouth of the estuary with the net flow. If, for example, the estuary flushing time is 10 tidal cycles, 10 tidal cycles following introduction into the estuary, a single batch of the pollutant has been essentially flushed out to the ocean. However, during each of the intervening tidal cycles, more of the pollutant enters the estuary and smaller portions of these batches are also flushed out. Eventually, a steady-state condition is reached in which a certain amount of pollutant enters the estuary, and the same amount is flushed out of the estuary during each tidal cycle. The amount of this pollutant which then resides in the estuary is a function of the flushing time. From the definition of flushing time, the amount of fresh water in the estuary may be calculated by:

$$W_E = T_F R \qquad \text{(VI-12)}$$

where

W_E = quantity of fresh water in the estuary

T_F = estuary flushing time

and

R = river discharge over one tidal cycle

Using the same approach, the quantity of fresh water in any segment of the estuary is given by:

$$W_i = T_i R \tag{VI-12a}$$

where

W_i = quantity of fresh water in the i^{th} segment of the estuary

and

T_i - flushing time for the i^{th} segment calculated by the fraction of fresh water method

If a conservative pollutant enters an estuary with the river flow it can be assumed that its steady-state distribution will be identical to that of the river water itself. Thus,

$$W_{pi} = W_i c_{pr} = T_i R c_{pr} \tag{VI-13a}$$

and

$$c_{pi} = W_{pi}/vol_i \tag{VI-13b}$$

where

W_{pi} = quantity of pollutant in estuary segment "i"

c_{pr} = concentration of pollutant in the river inflow

c_{pi} = concentration of pollutant in estuary segment "i" assuming all of pollutant "i" inflows to the estuary with the river discharge. Thus direct discharges into the estuary are now being disregarded.)

and

vol_i = water volume segment "i"

The same values for c_{pi} and W_{pi} may also be obtained by using the fraction of fresh water, f_i, for each segment by:

$$c_{pi} = f_i \, c_{pr} \qquad \text{(VI-14a)}$$

and

$$W_{pi} = c_{pi} \, vol_i \qquad \text{(VI-14b)}$$

Thus both the quantity of a pollutant in each segment and its concentration within each segment are readily obtainable by either of the above two methodologies. The use of one of these methods will be demonstrated in Example VI-5 below for calculation of both c_{pi} and W_{pi}.

It should also be noted that the total amount of a pollutant in residence in an estuary may be estimated by summing the segment W_{pi}'s by:

$$W_{pE} = \sum_{i=1}^{n} W_{pi}$$

EXAMPLE VI-5

Calculation of Concentration of Conservative River Borne Pollutants in an Estuary

For this example The Alsea Estuary again will be used since its flushing characteristics have already been determined. Assume that the State Department of Environmental Quality has just released that a planned expansion of upstream agricultural activity will result in a summer mean river concentration of 20 μg/l of a certain pesticide*. The planner must now calculate the steady-state concentration of pesticide throughout Alsea Estuary so that local biologists may determine if this activity will affect estuarine clam beds.

From previous calculations, $R = 4.64 \times 10^6$ ft^3tidal cycle. Converting pesticide concentrations into appropriate units:

$$c_{pr} = (20\ \mu g/l)\ (28.32\ l/ft^3) = 566.4\ \mu g/ft^3$$

Using the segmentation scheme derived for the fraction of fresh water flushing calculation, the tabular calculations shown in Table VI-4 can be made for c_{pi} and W_{pi}

As a check on this result, W_{pE} can alternately be calculated by

$$W_{pE} = T_f\ R\ c_{pr}$$

using the fraction-of-fresh-water method T_f as a check,

$$W_{pE} = (20.8)\ \text{tidal cycles}\ (4.64 \times 10^6\ ft^3/\text{tidal cycle})\ (566.4\ \mu g/ft^3)$$

or

$$W_{pE} = 54.67 \times 10^9\ \mu g$$

W_{pE} is within 0.41% of the previous estimate.

---

*Pesticides actually tend to absorb to particulates and settle to the sediment layer. However the assumption made here that they act conservatively will not create excessive error.

TABLE VI-4

CALCULATION OF SEGMENT CONSERVATIVE POLLUTANT QUANTITY FOR ALSEA ESTUARY

segment	f_i	c_{pi}		vol_i (10^7 ft^3)	W_{pi} (µg)
		µg/ft^3	µg/l		
1 (head)	.947	536.4	18.9	3.59	19.26×10^9
2	.856	484.8	15.8	1.32	6.40×10^9
3	.762	431.6	15.2	2.30	9.93×10^9
4	.638	361.4	12.8	2.45	8.85×10^9
5	.479	271.3	9.6	1.37	3.72×10^9
6	.315	178.4	6.3	1.06	1.89×10^9
7	.176	99.7	3.5	1.28	1.28×10^9
8 (mouth)	.0794	45.0	1.6	7.92	3.56×10^9
					$W_{pE} = 54.89 \times 10^9 \mu g = 121.0$ lbs

END OF EXAMPLE VI-5

In this example low tide volumes were used to calculate W_{pi} since low tide volumes had been used to calculate f_i's. The approach assumes that 1) c_{pi}'s are constant over the tidal cycle and that 2) W_{pi}'s are constant over the tidal cycle. This leads to the assumption that calculation of a low tide c_{pi} and W_{pi} will fully characterize a pollutant in an estuary. This, however, is not strictly true. Figure VI-23 depicts one tidal cycle in an estuary and shows the periods of the cycle during which a pollutant is flushed out of the estuary and during which river discharge brings pollutants into the estuary. During periods of high tide, rising tidal elevation blocks river discharge and backs up river flow into the lower stretches of the river. Figure VI-23 also shows the resulting quantity of a pollutant in residence in the estuary (W_{pE}) over the tidal cycle. This variation over the tidal cycle as a percentage of W_{pE} is dependent on the flushing time but is usually small (approximately 1.5% for Alsea Estuary). The change in the total volume of water in an estuary over a tidal cycle is equal to the tidal prism which is often of the same magnitude as the low tide volume. For Alsea Estuary, $P_t = 5.1 \times 10^8$ ft^3 while $V = 2.1 \times 10^8$ (Goodwin, Emmet, and Glenne, 1970). Thus the variation in estuarine volume is 2.5 times the low tide volume. As a result estuarine volume variations over a tidal cycle have a much greater impact on variations in pollutant concentrations in the estuary than do changes in the quantity of pollutant present in the estuary over a tidal cycle. It is important to note, however, that low tidal volume and low W_{pE} nearly coincide, so that variations in mean pollutant concentrations are less severe than are estuarine water mass changes.

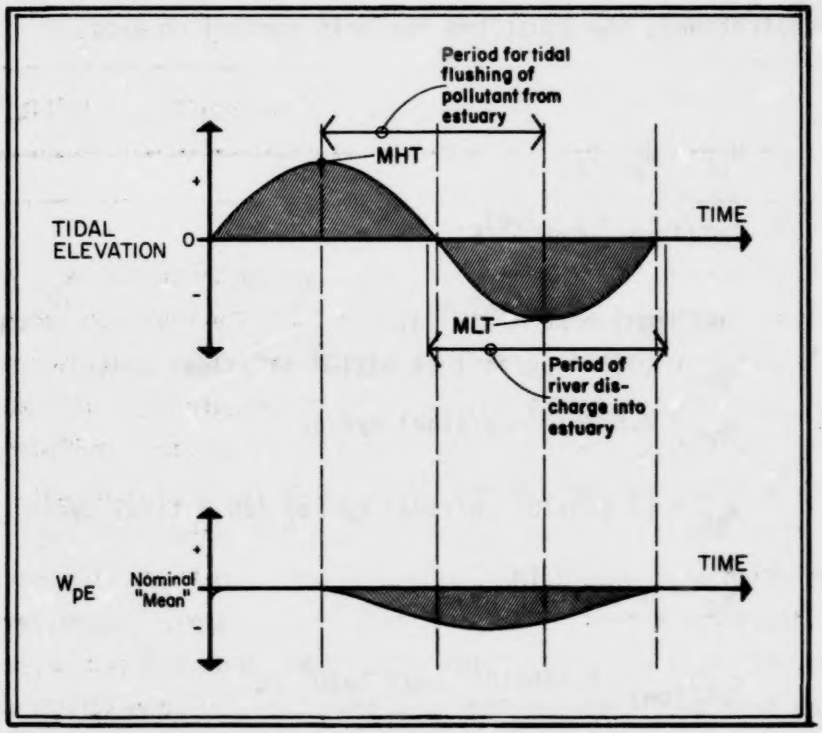
Period for tidal flushing of pollutant from estuary
MHT
+
TIDAL ELEVATION
0
-
TIME
MLT
Period of river discharge into estuary
+
W_{pE}
Nominal "Mean"
-
TIME

FIGURE VI-23 RIVER BORNE POLLUTANT CONCENTRATION FOR ONE TIDAL CYCLE

This qualitative description of pollutant flow into and out of an estuary is somewhat simplistic since it assumes that high tide and low tide at the mouth of an estuary will coincide with those at the head of the estuary. This is usually not the case. There is normally a significant lag time between tidal events at an estuarine mouth and those at it's head. Thus river discharge into the estuary which depends on tidal conditions at the head, and tidal discharge which depends on tidal conditions at the mouth, are not as directly tied to each other as indicated on Figure VI-23.

While W_{pE} does not vary substantially over a tidal cycle under steady state conditions, the mean concentration of a pollutant in an estuary (c_{pE}) will. Alsea Estuary data can be used to show this c_{pE} variation over a tidal cycle. Using data for the estuary as a whole (mean concentration), the equations for this comparison are:

$$W_{pE} = W_{pr}\, T_F, \qquad \text{(VI-15)}$$

$$c_{pE(low)} = W_{pE}/vol_E,$$

and $$c_{pE(high)} = W_{pE}/(vol_E + P_T) \qquad \text{(VI-16)}$$

with $$W_{pr} = (566.4\ \mu g/ft^3)\ (4.64\times10^6\ ft^3/\text{tidal cycle})$$

or $$W_{pr} = 2.628\times10^9\ \mu g/\text{tidal cycle}.$$

Then, $$W_{pE} = (2.628\times10^9\ \mu g/\text{tidal cycle})\ (20.8\ \text{tidal cycle})$$

$$W_{pE} = 5.466\times10^{10}\ \mu g,$$

and $$c_{pE(low)} = 5.466\times10^{10}\ \mu g/2.1\times10^8\ ft^3$$

or $$\underline{c_{pE(low)} = 260.31\ \mu g/ft^3}, \text{ or 46\% of river concentration.}$$

However, $$c_{pE(high)} = 5.466\times10^{10}\ \mu g/(2.1\times10^8\ ft^3 + 5.1\times10^8\ ft^3)$$

$c_{pE(high)} = 75.92\ \mu g/ft^3$, or 13% of river concentration

In an actual estuary the concentration of a pollutant is not a stepwise function as indicated by segment c_{pi} values, but is more realistically a continuous spectrum of values. By assigning the longitudinal centroid of each segment a concentration value equal to that segment's c_{pi}, a resulting continuous curve can be constructed as shown in Figure VI-24. This type of plot is useful in estimating pollutant concentrations within the estuary. It can also be used, however, to estimate maximum allowable c_{pr} to maintain a given level of water quality at any point within the estuary. This latter use of Figure VI-24 is based on determining desired c_{px} levels and then using the ratios of c_{px} to c_{pr}, to calculate an allowable c_{pr}.

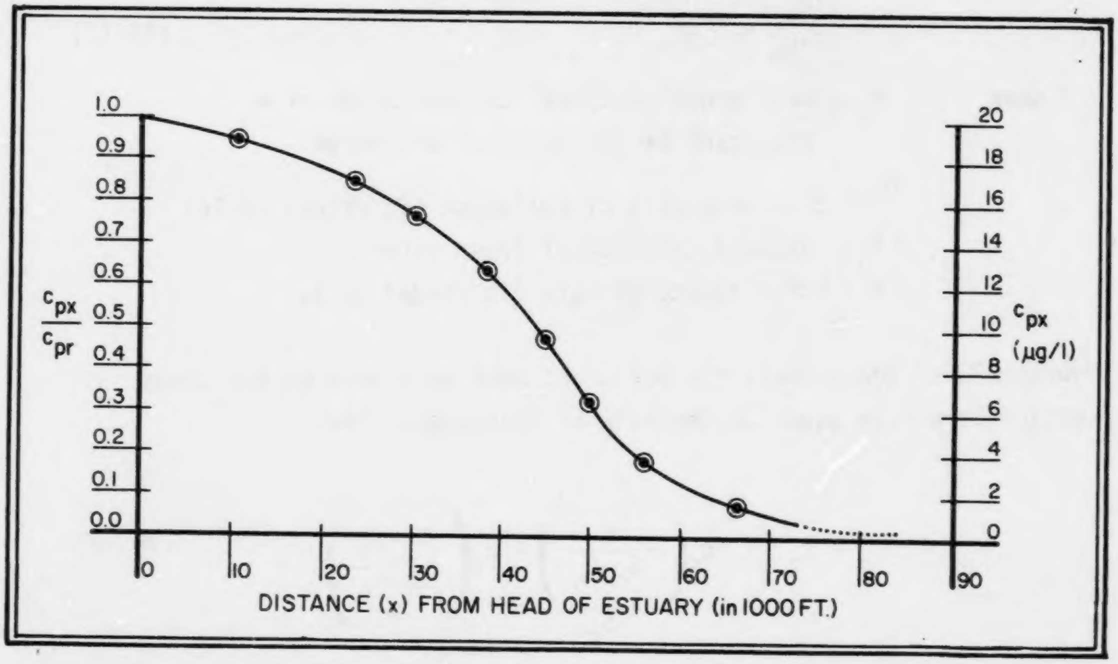

FIGURE VI-24 ALSEA ESTUARY RIVERBORNE CONSERVATIVE POLLUTANT CONCENTRATION

6.5.4. Other Continuous Conservative Pollutant Inflows

In the previous section an analysis was made of the steady state distribution of a continuous flow pollutant entering at the head of an estuary. The result was a graph of the longitudinal pollutant concentration within the estuary (Figure VI-24). This section will address a continuous, conservative pollutant flow entering along the side of an estuary. Such a pollutant flow (e.g. the conservative elements of a municipal sewer discharge, industrial discharge, or minor tributary) will be carried both upstream and downstream by tidal mixing with the highest concentration in the vicinity of the outfall. Once a steady state has been achieved, the distribution of this pollutant will be directly related to the distribution of fresh river water (Dyer,1973).

The average cross sectional concentration at the outfall under steady state conditions is:

$$c_o = \frac{Q_p}{R+Q_p} f_o \simeq \frac{Q_p}{R} f_o \qquad \text{(VI-17)}$$

when c_o = mean cross sectional concentration of a pollutant at the point of discharge.

Q_P = discharge rate of pollutant (ft^3/tidal cycle)

f_o = segment fraction of fresh water

R = river discharge rate (ft^3/tidal cycle)

Downstream of the outfall the pollutant must pass through any cross section at a rate equal to the rate of discharge. Thus,

$$c_x = c_o \frac{f_x}{f_o} = c_o \left(\frac{\frac{S_s - S_x}{S_s}}{\frac{S_s - S_o}{S_s}} \right) = c_o \left(\frac{S_s - S_x}{S_s - S_o} \right) \qquad \text{(VI-18)}$$

where s_x, c_x and f_x denote any downstream cross section.

and s_o, c_o and f_o denote the cross section at the discharge point (or segment into which discharge is made)

Upstream of the outfall, the quantity of pollutant diffused and advectively carried upstream will be balanced by that carried downstream by the non-tidal flow so that the net pollutant transport through any cross-section is zero. Thus, the pollutant distribution will be directly proportional to salinity distribution and, (Dyer, 1973):

$$c_x = c_o \frac{S_x}{S_o} \qquad \text{(VI-19)}$$

Downstream of the outfall, the pollutant concentration resulting from a point discharge will be directly proportional to river borne pollutant concentration. Upstream from the discharge point it will be inversely proportional to river borne pollutant concentrations. Figure VI-25 is a graph of f_x versus distance from the estuary head for a typical estuary. This solid f_x line is also a measure of pollutant concentration for all points downstream of a pollutant outfall (either discharge location A or B). The actual concentration (c_{px}) for any point is equal to this f_x value multiplied by Q_p/R which is a constant over all x. Upstream concentrations decrease from c_o in a manner proportional to upstream salinity reduction (see dotted line). It is important to note how even a small downstream shift in discharge location will create a very significant reduction in upstream steady state pollutant concentration.

EXAMPLE VI-6

Calculation of Estuarine Pollutant Concentrations for Varying Discharge Locations

A new industrial complex is to be built on the south shore of Alsea Estuary. Two outfall locations (A and B) are being considered. A is 18,5000 feet from the mouth, B is 31,000 feet. It must now be determined what the differential impact of discharges from these two points will be on each of three important ecological areas (1, 2, and

3). Area 1 is centered 7,000 feet from the estuary mouth, area 2 is centered 28,000 feet from the mouth and area 3 is centered 50,000 feet above the mouth. The primary concern is with the heavy metal content of the discharge. The concentration is listed as 800 μg/l in a 2.5 MGD discharge flow, and the pollutant may be assumed to act as a conservative constituent within the estuary.

First, longitudinal f_x and S_x values must be plotted. Figure VI-18 showed Alsea Estuary low river flow salinities, and Figure VI-24 showed Alsea Estuary f_x values. These are repeated in Figure VI-26. Additionally, both potential outfalls and the three study sites are marked in Figure VI-26.

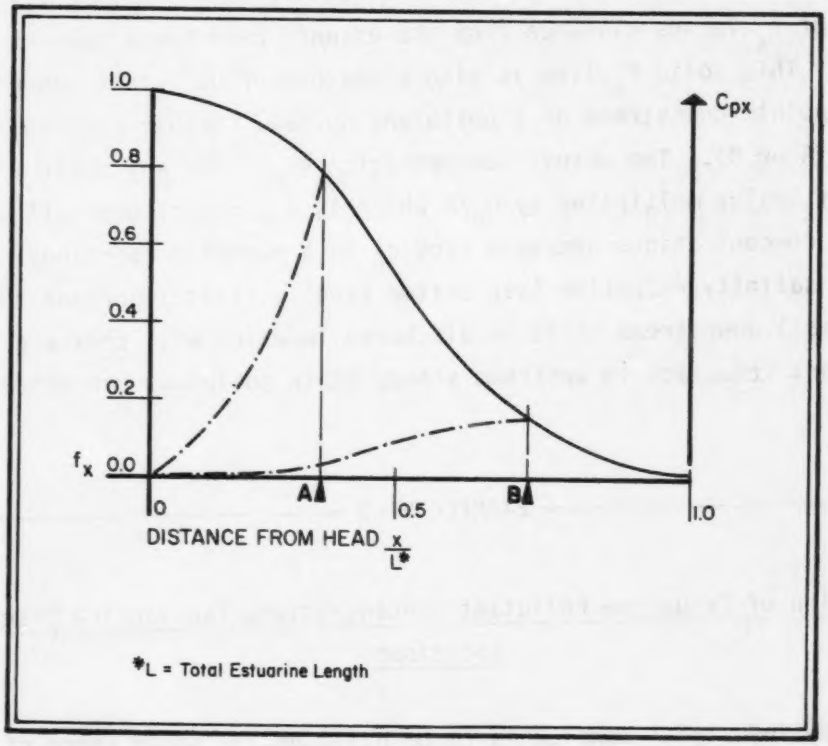

FIGURE VI-25 POLLUTANT CONCENTRATION FROM AN ESTUARINE OUTFALL

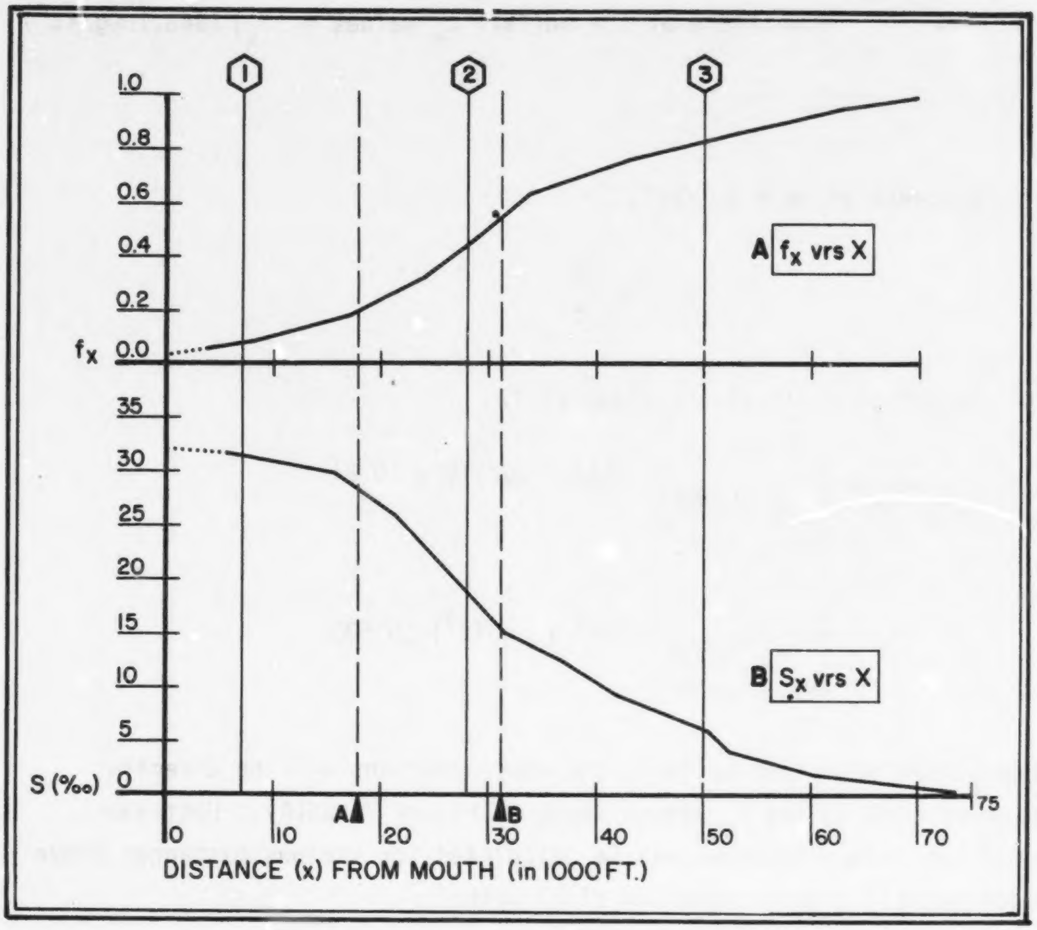

FIGURE VI-26 ALSEA ESTUARY SALINITY AND f_X PROFILE

c_{px} values were obtained by multiplying corresponding f_x values by Q_p/R as follows

$$Q_p = (800\ \mu g/l)\ (28.32\ l/ft^2)\ (1.54723\ cfs/mgd)$$
$$(2.5\ mgd)\ (3600\ sec/hr)\ (12.4\ hr/tidal\ cycle)$$

or $$Q_p = 3.912\text{x}10^9\ \mu g/tidal\ cycle$$

and $$\frac{Q_p}{R} = \frac{3.912\text{x}10^9\ \mu g}{4.64\text{x}10^6\ ft^3} = 843.1\ \mu g/ft^3$$

thus downstream of the outfall c_x values = (f_x) (843.1 μg/ft^3)

Upstream of each outfall,

$$c_x = c_o \frac{S_x}{S_o}$$

From Fig. VI-25 and Equation VI-17,

$$c_A = \frac{Q_p}{R} f_{(x=18,000)} = (843.1\ \mu g/1ft^3)\ (0.2)$$
$$= 168.6\ \mu g/ft^3$$

$$c_B = \frac{Q_p}{R} f_{(x=31,000)} = (843.1\ \mu g/ft^3)\ (0.53)$$
$$= 446.8\ \mu g/ft^3$$

Downstream of either outfall, the concentrations will be directly proportional to the f_x values shown on Figure VI-26(A). Upstream pollutant concentrations may be calculated for various distances above each outfall site by equation VI-17 with:

$$S_o/A = S_A = 28.3\ ^o/oo$$
$$S_o/B = S_B = 15.9\ ^o/oo$$

These results are shown in Table VI-5.

Assuming that the proposed outfalls were installed, the concentrations estimated using the longitudinal mean cross sectional concentration for heavy metals in Alsea Estuary are as shown in Figure VI-27. The resulting concentrations at each of the critical habitat areas are shown in Table VI-6.

TABLE VI-5

CONSERVATIVE POLLUTANT CONCENTRATIONS ABOVE AN ESTUARINE OUTFALL

	OUTFALL A		OUTFALL B	
Distance above outfall (ft)	c_x(µg/ft^3)	x^*	c_x(µg/ft^3)	x^*
5,000	144.2	23,000	365.3	36,000
10,000	113.2	28,000	309.1	41,000
15,000	85.5	33,000	219.2	46,000
20,000	70.9	38,000	154.6	51,000
25,000	52.7	43,000	88.5	56,000
30,000	41.7	48,000	56.2	61,000
35,000	23.3	53,000	35.1	66,000
40,000	16.1	58,000	19.7	71,000

x^* = distance from mouth in feet

TABLE VI-6

COMPARISON OF POLLUTANT CONCENTRATIONS FOR ALTERNATE OUTFALLS IN ALSEA ESTUARY

outfall used / critical habitat number	mean cross section concentration A		B	
	µg/ft^3	µg/l	µg/ft^3	µg/l
1	62	2.19	62	2.19
2	112	3.95	370	13.07
3	32	1.13	166	5.86

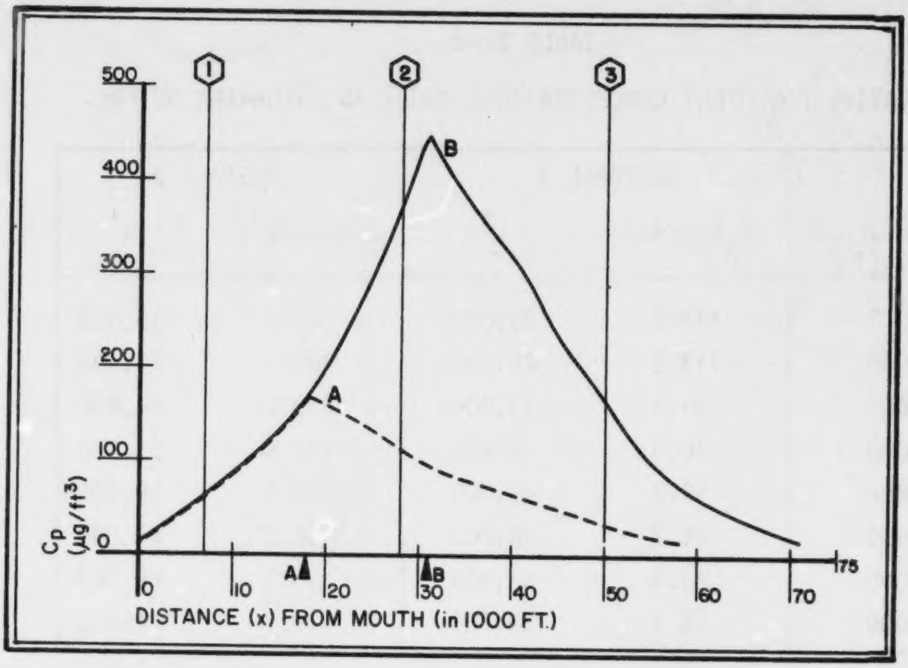

FIGURE VI-27 ALSEA ESTUARY EXAMPLE HEAVY METAL CONCENTRATION FOR TWO OUTFALLS

The major advantage of using discharge points nearer to the estuary's mouth is not in improvements in downstream water quality but in the large upstream improvements in water quality. Thus, it is advantageous to locate estuarine outfalls as near the mouth of the estuary as possible.

END OF EXAMPLE VI-6

6.5.5 Continuous Flow Non-Conservative Pollutants

Most of the discharges made into an estuary will have non-conservative components, with BOD being the most common of these. In addition to dispersion and tidal mixing, these components will also decay as a function of time. The concentration of the non-conservative pollutants will thus always be lower than that for conservative pollutants (which have a decay rate of zero) for equal concentration of discharge rates. The results of the previous section for conservative constituents will serve to set upper limits for the concentration of non-conservative continuous flow pollutants. Thus, if plots similar to Figure VI-24 for river discharges and to Figure VI-27 for other direct discharges have been prepared for flow rates equal to that of the non-conservative pollutant under study, some reasonable approximations can be made for steady state non-conservative pollutant concentrations without requiring additional data. Assuming a first order decay rate for non-conservative constituents, non-conservative concentration is given by:

$$c_t = c_o e^{-kt} \qquad \text{(VI-20)}$$

where c_t = pollutant concentration at time "t",

c_o = initial pollutant concentration

k = decay or reaction constant

and t = elapsed time (usually measured in days or tidal cycles)

For conservative pollutants k = 0 and $c_t = c_o$ under steady state conditions. k values are determined empirically, and depend on a large number of variables. Typical k values for the two major estuarine decay phenomena to be addressed (BOD and coliform bacteria) are shown in Table VI-7. If data are not available for a particular estuary, the use of these average values will at least ensure reasonable estimates.

TABLE VI-7

TYPICAL VALUES FOR DECAY REACTION RATES 'k'*

Source	BOD	coliform
Dyer, '73		.578
Ketchum, '55		.767
Chen and Orlob '75	.1	.5
Hydroscience '71	.05-.125	1-2
McGaughey '68	.09	
Harleman '71	.069	

*k values for all reactions given on a per tidal cycle basis.

It should be noted that decay rates are dependent upon temperature. The values given assume a temperature of 20° C. Variations in k values for differing temperatures are given by equation VI-21.

$$k_T = k_{20^\circ} \Theta^{T-20^\circ} \qquad \text{(VI-21)}$$

where

k_T = decay rate at temperature "T",

k_{20} = decay rate at 20°C (as given in table VI-7),

and

Θ = a constant (normally between 1.03 and 1.04)

Thus an ambient temperature of 10° C would reduce a k value of 0.1 per tidal cycle to 0.074 for a Θ=1.03. Table VI-7A shows the temperature correction for various temperatures. The actual k value is then the product of the k_{20^o} value and the correction factor. Where temperature data are not available for either an estuary as a whole or for a segment of the estuary, use k_{20^o} values.

TABLE VI-7A

TEMPERATURE CORRECTION FOR SELECTED TEMPERATURES

Temperature (^{o}C)	$\Theta^{T-20°*}$
6	0.661
8	0.701
10	0.744
12	0.789
14	0.838
16	0.889
18	0.943
20	1.000
22	1.061
24	1.126
26	1.194
28	1.267
30	1.344

*for $\Theta = 1.03$

Decay effects can be compared to flushing effects by setting time equal to the flushing time and comparing the resulting decay to the known pollutant removal as a result of flushing.

If kt in equation VI-20 is less than 0.5 for $t = T_f$, decay processes would reduce concentration by only 1/3 over the flushing time. Here mixing and advective effects dominate and non-conservative decay can be discounted. When $kT_f > 12$ decay effects would reduce a batch pollutant to 5% of its original concentration in less than 1/4 of the flushing time. In this case, decay processes dominate and steady state concentrations will approximate those resulting from decay alone. Between these extremes, both processes are active in removing a pollutant from the estuary with $3 < kT_f < 4$ being the range for approximately equal contributions to removal. Dyer (1973) analyzed the situation for which decay and tidal exchange are of equal magnitude for each estuarine segment.

Knowing the conservative concentration, the non-conservative steady-state concentration in a segment is given by:

$$c_i = c_o \frac{f_i}{f_o}\left(\frac{r_i}{1-(1-r_i)e^{-k}}\right)^n \text{ for segments downstream of the outfall} \qquad \text{(VI-22a)}$$

and

$$c_i = c_o \frac{S_i}{S_o}\left(\frac{r_i}{1-(1-r_i)e^{-k}}\right)^n \text{ for segments upstream of the outfall} \qquad \text{(VI-22b)}$$

where c_i = non-conservative constituent mean concentration in segment "i",

c_o = conservative constituent mean concentration in segment of discharge,

r_i = the exchange ratio for segment "i" as defined by the Modified Tidal Prism Method,

n = number of segments away from the outfall (i.e. n=1 for segments adjacent to the outfall; n=2 for segments next to these segments, etc.)

and other parameters are as previously defined.

EXAMPLE VI-7

Continuous Discharge, Non-Conservative, Steady State Concentration Calculation

Example VI-6 considered an agricultural runoff containing pesticides. The flow entered Alsea Estuary from the river. In addition to pesticide concentrations, however, BOD concentrations in an estuary as a result of projected agricultural activity should be calculated.

Based on DEQ data and previous calculations for river flow (see Chapter 4 for river related calculations) it is known that the summer mean river concentrations of BOD resulting from this activity as it enters Alsea Estuary is 3.2 mg/l.

It is reasonably apparent that decay and tidal mixing for BOD in Alsea Estuary are both important.

From previous calculations, $T_f \simeq 17$ tidal cycles and thus it can be estimated that $kT_f \simeq 1.7$ (see Table VI-7 for k values). For a kT_f value in this range it would be expected that tidal action predominates but that decay significantly reduces steady state concentrations.

Values of f_x and S_x for Alsea Estuary are shown on Figure VI-26. Since the current problem involves river borne introduction of pollutants, there are no upstream segments in the estuary and salinity values will not be needed. Values of r_i and segment designations are obtained from the Modified Tidal Prism Method flushing time calculations. Table VI-8 summarizes this information and lists B_i values where:

$$B_i = \left(\frac{r_i}{1-(1-r_i)e^k} \right)^n$$

thus $$c_i = c_o \frac{f_i}{f_o} B_i$$

Given that $c_R = 3.2$ mg/l,

then $$c_o = c_r\, f_o = (3.2 \text{ mg/l})\ (.970) = 3.104 \text{ mg/l}$$

This c_o value is for conservative pollutants.

Thus

$$c_o = c_o \frac{f_o}{f_o} B_o = c_o B_o = (3.104\ \text{mg/l})\ (.9096) = 2.82\ \text{mg/l}$$

for non-conservative pollutants.

TABLE VI-8

TABULATION OF VALUES FOR NON-CONSERVATIVE POLLUTANT CONCENTRATION

segment	n	f_i	r_i	B_i^*
0	1	.970	.489	0.9096
1	2	.921	.707	0.9256
2	3	.805	.450	0.7189
3	4	.547	.663	0.8278
4	5	.119	.800	0.8891

*k= 0.1 is assumed for these calculations

Having calculated segment '0' concentration, remaining segments are estimated by table VI-8 and:

$$c_i = c_o \frac{f_i}{f_o} B_i$$

segment	c_i (mg/l)
0	2.82
1	2.48
2	1.68
3	1.32
4	0.31

Figure VI-28 shows this estimate of non-conservative concentration distribution. Additionally a plot of the concentration of a conservative pollutant with the same discharge characteristics is shown for comparison. Large segment exchange ratios result in lower amounts of pollutant decay as residence time in the segment is reduced. The mass of pollutant decayed with time increases directly with the decay rate k. To demonstrate this, the dotted line on Figure VI-28 uses a k value of 0.578 (typical for coliform bacteria decay). The initial discharge rate is equal to the previous BOD discharge rate. However, an increased decay rate results in concentration in the mid-estuary section of approximately 1/3 the BOD concentration. Changes in mean section DO levels could now qualitatively be estimated as the inverse of the change in BOD load throughout the estuary. An example of this inverse relationship is shown in Figure VI-29. As can be seen, there is longitudinal translation of peak BOD to peak DO deficit due to net downstream advection. Additionally peak BOD is greater than peak DO deficit. The concept of DO deficit was introduced in previous chapters of this report and will not be expanded here. Because of the complexities of estuarine transport and of the complete oxygen balance, and because many water quality regulations are now written for maximim allowable BOD, this chapter will not go beyond BOD calculation. Where large BOD loads exist (5 mg/l or more), DO problems may be inferred. A complete analysis of the oxygen balance in that spatial area should then be conducted using computer model tech-

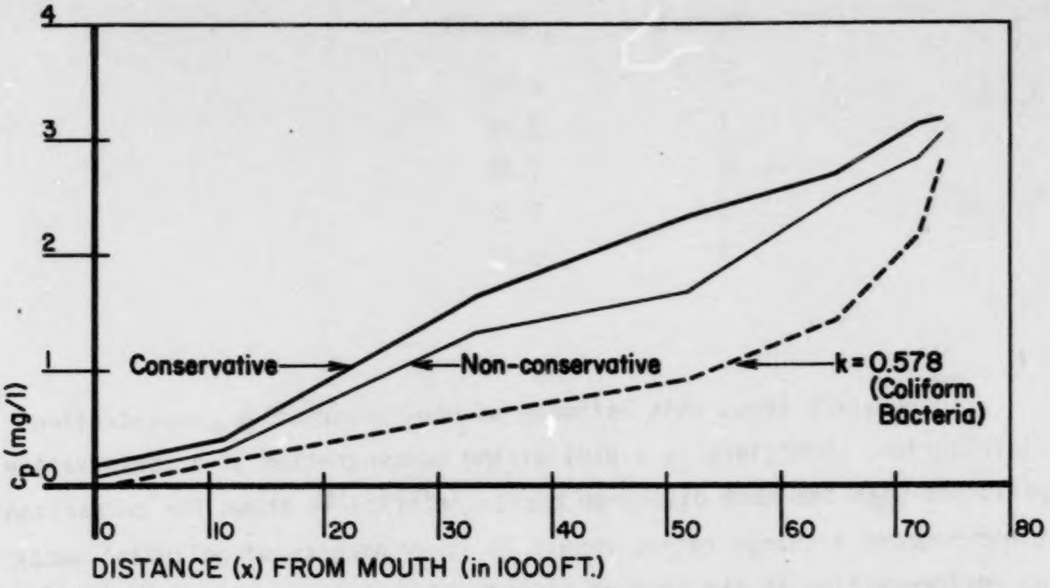

FIGURE VI-28 EXAMPLE ESTUARINE NON-CONSERVATIVE POLLUTANT CONCENTRATION

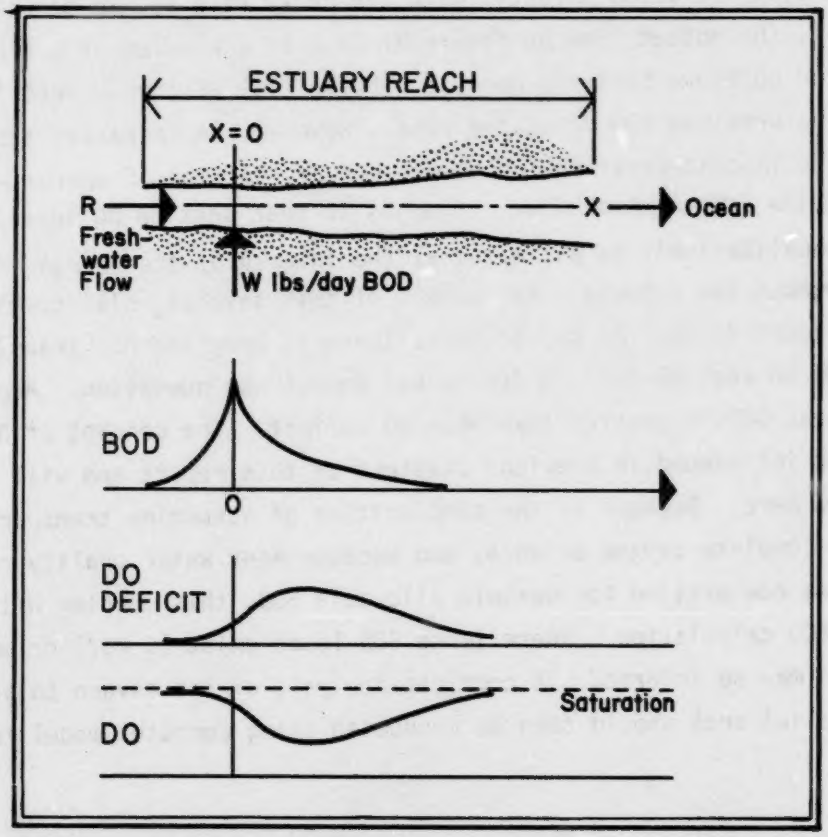
ESTUARY REACH
X=0
R
Fresh-water Flow
X
Ocean
W lbs/day BOD
BOD
0
DO DEFICIT
Saturation
DO

FIGURE VI-29 INCREMENTAL BOD LOADING - DO RESPONSE IN ESTUARIES

niques. The principle goal here is screen large estuarine areas and to identify probable water quality problem areas. A BOD analysis is adequate for this purpose.

END OF EXAMPLE VI-7

6.5.6 Use of Dispersion Equations

Until now the distributions of salinity and fresh water within an estuary have been used as an indicator of the steady state results of dispersion. Without knowledge of roughness coefficient, hydraulic radius, and actual dispersion coefficients, this procedure is as good an approximation as is practicable. If necessary data are available, however, the use of a more formal dispersion equation for non-conservative pollutants is advisable.

Good reviews of this approach are provided by Harleman (1971), O'Connor and Thomann (1971), and in summary form by Hydroscience (1971). The approach and equational forms used by each researcher is the same.

The basic steady state equation of mass conservation for non-conservative substances in an estuary is:

$$0=E\frac{d^2c}{dx^2} - U\frac{dc}{dx} - kc \quad \text{(O'Connor and Thomann, 1971)} \qquad \text{(VI-23)}$$

where E = diffusion coefficient,

c = substance concentration

U = net velocity

and k = decay, or reaction, coefficient as previously defined.

The solution for equation VI-23 for the concentration at any point is:

$$c = c_o e^{-gx} \text{ for } x<o \quad \text{(downstream)}$$

$$\text{and} \quad c = c_o e^{jx} \text{ for } x>o \quad \text{(upstream)} \qquad \text{(VI-24)}$$

where: c_o is the concentration at the outfall point and is given by

$$c_o = \frac{w}{R\sqrt{1+\frac{4kE}{U^2}}} = \frac{w}{A\sqrt{U^2+4kE}} \qquad \text{(VI-25)}$$

w = waste discharge load
A = cross sectional area,

$$g = \frac{1}{2E}\left[U+\sqrt{U^2+4kE}\right] = \frac{U}{2E}\left[1+\sqrt{1+\frac{4kE}{U^2}}\right] \qquad \text{(VI-26)}$$

and

$$j = \frac{1}{2E}\left[U-\sqrt{U^2+4kE}\right] = \frac{U}{2E}\left[1-\sqrt{1+\frac{4kE}{U^2}}\right] \qquad \text{(VI-27)}$$

The major unknowns in this set of equations are k and E. As was covered in the previous section, k is an empirical approximation of actual decay, consumption and mortality processes and can be approximated by a range of values. E, the dispersion coefficient, is also an empirical coefficient, and can be explicitly determined for any estuary only by direct in-situ measurement. However, Harleman (1964) has developed a formulation for approximating actual dispersion coefficient values. This is:

$$E = 77\, nv\, R_h^{5/6} \qquad \text{(VI-28)}$$

where: n = Mannings roughness coefficient*
v = velocity in fps
R_h = hydraulic radius in feet**
and E = dispersion coefficient in ft^2/sec

---

[*] This coefficient is a function of the bottom surface. Typical estuarine values range from 0.028 to 0.035 depending on bottom cover.

[**] The hydraulic radius is defined as the cross sectional area divided by the wetted perimeter of that cross section.

Harleman (1971) discussed this approach to estimating E and was able to relate E to maximum tidal velocity by:

$$E = 0.3\, n\, U_{max}\, R_h^{5/6} \qquad \text{(VI-29)}$$

where

E = longitudinal dispersion coefficient (in mi^2/day)
n = Manning roughness coefficient
U_{max} = maximum tidal velocity (in feet/sec)

and

R_h = hydraulic radius (in feet)

Equation VI-29 can be written for E_1 calculation in ft^2/sec units by:

$$E \approx 100\, n\, U_{max}\, R_h^{5/6} \qquad \text{(VI-29a)}$$

Use of either Equation VI-29 or VI-29a has the advantage of using maximum tidal velocity rather than net non-tidal flow. U_{max} is directly measurable and is generally available for U.S. estuaries in existing literature. Both of these equational forms assume constant density over a cross-section. While this limits the applicability of these E approximations, it is consistent with previous assumptions of this chapter for uniform cross-sectional U and S distribution.

The dispersion coefficient will vary somewhat over the length of an estuary, decreasing toward the head of the estuary. Evenson (1977, personal communication) has identified three principal components which comprise a dispersion coefficient: tidal mixing, salinity gradient mixing, and net fresh water advective flow. The relative location in an estuary for which each of these factors is significant, and their relative magnitude is shown in Figure VI-30. Compounding the variation in causal forces, the datum used as a basis for the calculations (MHT, MLT, tidally averaged) will result in very different estimates for E. Nonetheless, some range of values for certain types of estuaries can be obtained. Table VI-9 shows a range of values for kE for several qualitative types of estuaries compiled by Hydroscience (1971). Knowing that k values vary over a range of values, the range of potential values for E is expanded from the kE listed in Table VI-9. For example, if k varies from 0.2 to 0.5 for a major estuary, E could vary from 10 to 100 mi^2/day and still satisfy the listed kE range. Additionally, Table VI-10 lists E values calculated by Hydroscience (1971) for selected estuaries. As can be seen from Table VI-10 variations in dispersion parameters are large over the lengths of a typical estuary. Unless specific E values are available from field experimentation it is felt that the potential error from incorrect value selection is too large to justify the use of this approach. In a case in which the previous methods independent of a dispersion coefficient are inadequate and in which specific E values are not known, the use of computer models or dye studies to determine actual E values are recommended.

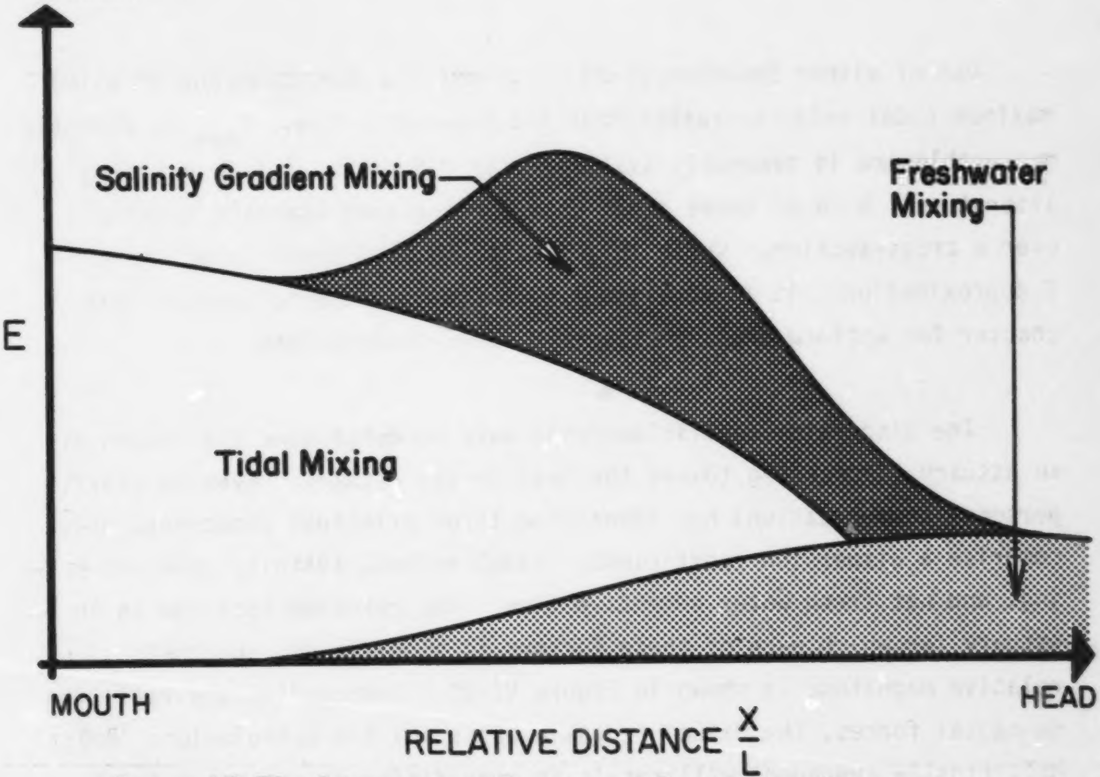

FIGURE VI-30 CONTRIBUTING FACTORS TO DISPERSION COEFFICIENTS IN THE ESTUARINE ENVIRONMENT

TABLE VI-9

kE VALUES FOR SEVERAL TYPES OF ESTUARIES

Estuary Description	kE (mi^2/day^2) Average Value	 Range
Large, deep main channel, major tidal action; near mouth of many estuaries.	10	5-20
Moderate navigation channel, large tidal tributaries; in upstream saline intrusions.	3	2-5

TABLE VI-10

E VALUES FOR SELECTED ESTUARY
(FROM HYDROSCIENCE 1971)

Estuary	R (cfs)	Low flow net non tidal velocity (fps) head - mouth	E (mi^2/day^*)
Delaware River	2,500	0.12-0.009	5
Hudson River (N.Y.)	5,000	0.037	20
East River (N.Y.)	0	0.0	10
Cooper River (S.C.)	10,000	0.25	30
Savannah R. (Ga., S.C.)	7,000	0.7-0.17	10-20
Lower Raritan R. (N.J.)	150	0.047-0.029	5
South River (N.J.)	23	0.01	5
Houston Ship Channel (Texas)	900	0.05	27
Cape Fear River (N.C.)	1,000	0.48-0.03	2-10
Potomac River (Va.)	550	0.006-0.0003	1-10
Compton Creek (N.J.)	10	0.01-0.013	1
Wappinger and Fishkill Creek (N.Y.)	2	0.004-0.001	0.5-1

* 1 $mi^2/day = 322.67\ ft^2/sec$

6.5.7 Multiple Waste Load Parameter Analysis

The preceding analysis allowed calculation of the longitudinal distribution of a pollutant, whether it is conservative or non-conservative, resulting from a single waste discharge. However, the planner will probably want to simultaneously assess both conservative and non-conservative elements from several separate discharges. This can be accomplished by graphing all desired single element distributions on one graph showing resultant concentration versus length of the estuary. Once graphed, the resulting concentration may be linearly added to obtain a total resultant incremental waste load.

An example of this procedure is shown in Figure VI-31. Note that the sum of the new discharges may be added to original ambient conditions to estimate new ambient concentrations along the length of the estuary. This technique of graphing outfall location and characteristics with resulting estuarine pollutant concentration should be done for all anticipated discharges. This will provide the planner with a good perspective on the source of potential water quality problems.

6.5.8 Graphical Analysis

In an effort to develop simplified planning guidelines Hydroscience (1971) applied the basic dispersion equations (Equation VI-24 through VI-26) over a wide range of tidal conditions for a dissolved oxygen analysis. The analysis covered the ranges of the basic variables which might be expected under normal estuarine conditions and allowed for variation in river discharge, net nontidal velocity, mean cross sectional area, section mean salinity, local dispersion coefficients and general estuarine topography. Additionally, water temperature, reaeration coefficients and surface oxygen transfer coefficients were included as major system variables to allow a more complete oxygen balance analysis.

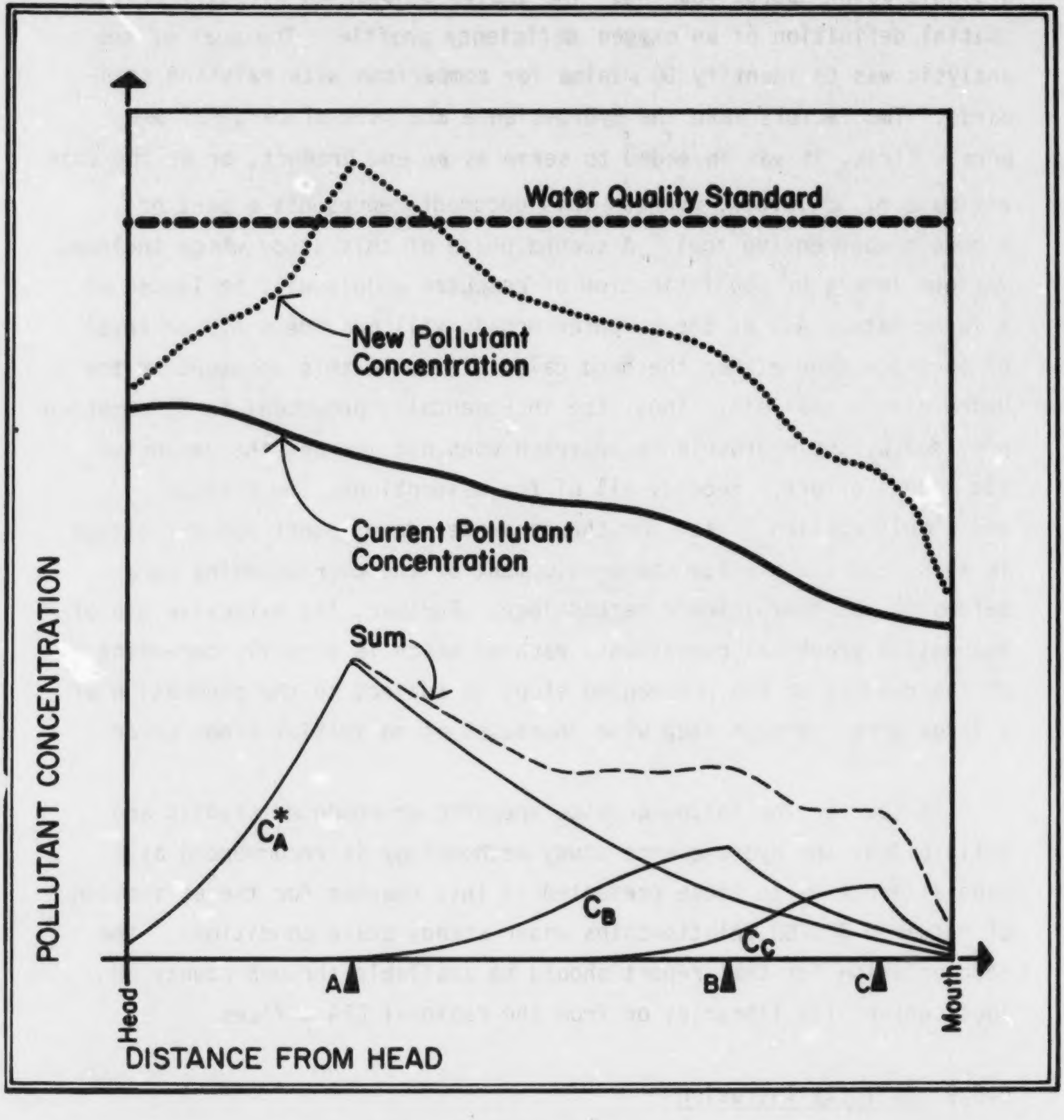

FIGURE VI-31 ADDITIVE EFFECT OF MULTIPLE WASTE LOAD ADDITIONS

The results of the Hydroscience analysis were a series of sequential graphs which could be used to estimate ultimate BOD, maximum oxygen deficit, and minimum estuarine mean DO levels as a function of a single point source loading. The analysis does not directly provide spatial definition of an oxygen deficiency profile. The goal of the analysis was to identify DO minima for comparison with existing standards. Two factors make the Hydroscience analysis of marginal use here. First, it was intended to serve as an end product, or as the sole estimate of DO levels, whereas this document represents a part of a more comprehensive tool. A second phase of this study which includes various levels of sophistication of computer models will be issued at a later date. All of the computer models will provide a higher level of accuracy than either the hand calculations in this document or the Hydroscience analysis. Thus, the incremental improvement in information provided by the Hydroscience approach does not warrant the extensive additional effort. Second, all of the assumptions, limitations, and simplification listed for the parameter development and definition in this report apply for the development of the corresponding parameters in the Hydroscience methodology. Further, the extensive use of sequential graphical operations, each of which is directly dependent on the results of the preceeding step, is subject to the generation of a large error through step wise increases of an initial minor error.

If however, no follow-on site specific or computer studies are anticipated, the Hydroscience study methodology is recommended as a good alternative to those presented in this chapter for the estimation of estuarie BOD-DO relationships under steady state conditions. The documentation for that report should be available through county or local university libraries or from the regional EPA offices.

6.5.9 Salinity Intrusion

Upstream intrusion of sea salt into the lower reaches of a river system is often of interest to local planners. Salt intrusion can endanger municipal water supplies, greatly reduce agricultural production in riparian lands, change sedimentation patterns and alter the spatial distribution of both fresh water and estuarine species. In addition to the seasonal changes in the salinity intrusion length previously mentioned, planned changes in either estuarine channel configuration or in river discharge will alter salinity intrusion patterns. Channel configuration changes are primarily associated with dredging projects. An increased cross sectional area results in increased advective upstream transport of bottom saline waters and in decreased net non-tidal velocities with the associated increased upstream salt diffusion. Planned river discharge reductions can result from increased agricultural fresh water withdrawals, additional municipal and industrial withrawals, water diversion programs or upstream water storage projects. The decreased river discharge will result in decreased net non-tidal velocities and thus in increased upstream diffusion transport.

In previous sections the steady state distribution of conservative pollutants was analyzed by assuming that this distribution was similar to the salinity distribution. If salt itself is now treated as a conservative pollutant, these same basic methdologies could be used to analyze a new unknown salinity distribution (Stommel, 1953). However, the methods presented for a continuous flow conservative pollutant are not applicable since they require a known salinity distribution.

Prediction of a new salinity intrusion length can be approached by use of the dispersion equations presented in the previous section (Equation VI-24 through VI-27). For conservative pollutants the form of Equation VI-27 reduces to:

$$J = \frac{Rx}{AE} = \frac{Ux}{E} \quad \text{(Stommel, 1953)} \qquad \text{(VI-30)}$$

This may now be applied to the new salinity regime by setting c_o at x = o (at the mouth of the estuary) equal to ocean surface salinity

($\approx 35^o/oo$) and by setting $1^o/oo$ as the upper end of the salinity intrusion. Thus from Equation VI-24

$$\frac{c_x}{c_o} = \frac{1^o/oo}{35^o/oo} = e^{-(Rx/AE)}$$

or $$3.5554 = \frac{Rx}{AE}$$

and

$$x = \frac{3.5554\ AE}{R} \qquad \text{(VI-31)}$$

where

x = length of the salinity intrusion at $1^o/oo$ or greater and where A, E, and R are as previously defined.

The solution of equation VI-31 is an iterative process. A and E will both vary with x. Thus, the procedure for use of VI-31 should be to assume an x value, obtain the A and E values associated with that x and then put the A and E values into equation VI-31 to see if the calculated x equals the assumed x. Based on the magnitude and direction of the resulting in-equality, select a new x value and repeat the procedure until the resulting error is acceptably small (in the range of 5% of x).

6.5.10 Non-Continuous (Single Event) Discharges

Single event discharges into an estuary are of much less concern to the planner than are continuous discharges. This is true for several reasons. First, single event discharges (storm drains, leaks, spills, etc.) normally represent a small fraction of the total annual estuarine waste load. Second, the effects of such single event discharges are relatively short-lived and will thus have a much smaller impact on the estuarine ecosystem. Within these constraints however, major single events (large storms, major oil spills, etc.) can have a significant short term impact on estuarine water quality.

Analysis of conservative and non-conservative substances will be approached separately as was the case for continuously discharged pollutants.

6.5.10.1 Conservative (Single Event) Pollutants

If a single event discharge enters an estuary with the river discharge, the flushing time provides a measure of the total estuarine residence time for that pollutant. As was evident from Figure VI-24 and Example VI-4, maximum pollutant concentration will occur near the head of the estuary. Methods described for calculation of concentrations of conservative pollutants are not applicable for single event phenomena since such events will not reach a steady-state condition. The concentration calculated by continuous flow methodology will, however, act as an upper limit to the concentration of a single event pollutant flow for a specified discharge rate. An additional estimation of single event concentration can be made based on the following assumptions:

1. Discharge period into the estuary equals one tidal cycle.
2. All pollutant entering the estuary during the one tidal cycle will remain in the inner-most estuarine segment throughout the cycle(i.e. the pollutant will obey "plug flow" theory).

The curve in Figure VI-32 represents the temporal flow of a pollutant into an estuary over one tidal cycle. Combining this with the two assumptions listed above, the following two estimates of pollutant concentration for the inner-most segment can be made.

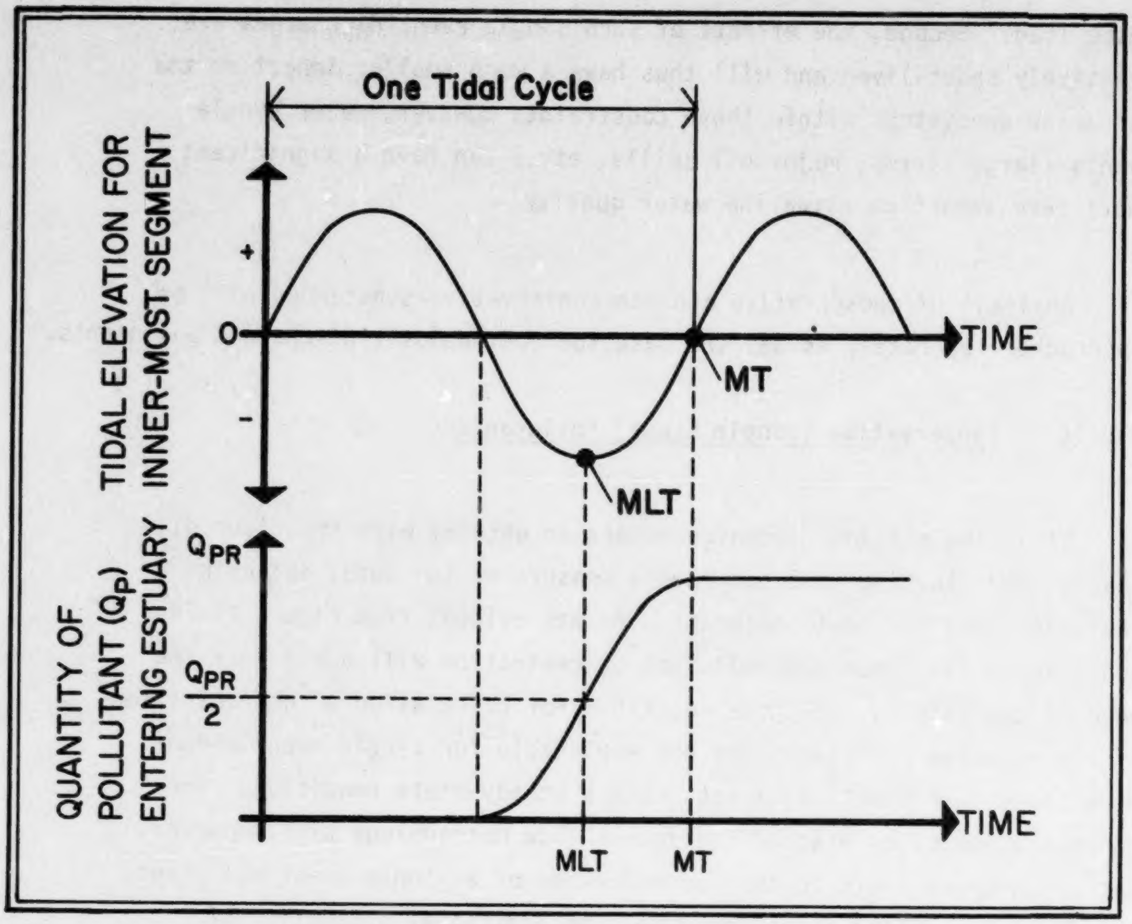

FIGURE VI-32* SINGLE EVENT POLLUTANT FLOW INTO AN ESTUARY

$$c_{Pi(MLT)} = c_{Pr} \quad \frac{R}{2 \cdot vol_i} \qquad \text{(VI-32a)}$$

and

$$c_{Pi(MT)} = c_{Pr} \quad \frac{R}{vol_i + P_{i/2}} \qquad \text{(VI-32b)}$$

where

$c_{Pi(MLT)}$ = concentration of pollutant in segment "i" at mean low tide of the first tidal cycle after pollutant entry

$c_{Pi(MT)}$ = concentration of pollutant in segment "i" at mean tide of the first cycle after pollutant entry

c_{Pr} = concentration of pollutant in river discharge

and R, vol_i and P_i are as previously defined.

The relative magnitude of these two concentrations will depend on the relative magnitudes of P_i and R. Normally $c_{Pi(MLT)}$ will be larger and will thus represent peak concentration of a conservative pollutant for this event.

Single event conservative pollutant flows entering either along the sides of an estuary or directly into the central waters are not as easy to analyze. A lower limit for estuarine residence time can be estimated by adding the segment flushing times for the segment of entry and all seaward segments. This, however, assumes that none of the pollutant is transported upstream from the segment of entry which is generally not true. This residence time and associated pollutant concentration is really a function of local dispersion characteristics, estuarine flushing characteristics, length of the discharge period, and the time of release with respect to tidal stages. A pollutant released during an outgoing tide will have very different residence and concentration characteristics than one released during an incoming tide. While it is obvious that a pollutant release during an ebb tide is preferable to one during a flood tide it is equally obvious that neither the time of storms nor that of leaks and spills is subject to discretionary planning.

Peak concentration for a single event discharge will be at the discharge point, or in the segment of discharge. By assuming that lateral and vertical dispersion is complete and instantaneous* and that the discharge occurs instantaneously, a peak pollutant concentration can be estimated by:

$$c_{Pi(max)} = Q_{Pi}/vol_i \qquad \text{(VI-33)}$$

where

i = segment of discharge

$c_{Pi(max)}$ = maximum potential concentration of the pollutant

Q_{Pi} = quantity of pollutant released

and

vol_i = low tide volume of segment i

---

*This appears to be a reasonable assumption, especially for verticle mixing. Work by Fisher (1968) estimated vertical mixing as:

$$E_Z = 0.23\ dU^*$$

where

E_Z = vertical dispersion coefficient

d = segment depth

and

U^* = shear velocity

Resulting E_Z values for typical estuaries are on the order of 0.2 to 1 ft/sec. Thus a 60 ft deep segment would be vertically mixed in approximately 1 minute - a very short period with respect to a tidal period.

Ippen (1966) applied finite difference techniques to the basic conservation of mass equation* in order to derive a stepwise method for tracking longitudinal concentration of such an instantaneous conservative discharge. Knowing an initial concentration in the segment of entry, this technique may be applied by:

$$c_{x,t+\Delta t} = c_{x,t}\left[1 - \frac{1}{A_x}\frac{\Delta t}{\Delta x^2}\left\{(AD')_{x+\Delta x/2} + (AD')_{x-\Delta x/2}\right\}\right] +$$

$$c_{x+\Delta x,t}\left[\frac{-U_x\Delta t}{2\Delta x} + \left\{\frac{(AD')_{x+\Delta x/2}}{A_x}\right\}\left\{\frac{\Delta t}{(\Delta x)^2}\right\}\right] +$$

$$c_{x-\Delta x,t}\left[\frac{U_x\Delta t}{2\Delta x} + \left\{\frac{(AD')_{x-\Delta x/2}}{A_x}\right\}\left\{\frac{\Delta t}{(\Delta x)^2}\right\}\right] \qquad \text{(VI-34)}$$

where

x = longitudinal distance from the discharge point

Δx and Δt = longitudinal and time steps

A_x = cross-sectional area at "x"

c = pollutant concentration

---

*This conservation of mass equation is:

$$\frac{\partial c}{\partial t} + U\frac{\partial c}{\partial x} = \frac{1}{A}\frac{\partial}{\partial x}\left(AD_x\frac{\partial c}{\partial x}\right) - Kc$$

where

D_x = longitudinal dispersion coefficient

A = cross-sectional area

c = pollutant concentration

U = current velocity

and

Kc = decay term which is disregarded in an analysis of conservative pollutants.

and

D´ = apparent longitudinal dispersion coefficient. D´ includes dispersion due to vertical instabilities as well as tidal and dispersion effects. In areas of little vertical instability (well mixed estuaries) verticle instability mixing is minimal and D approximately equals dispersion coefficients as previously defined in this chapter.

Application of Equation VI-34 requires that consecutive calculations be made for points x defined by $x = \pm n\Delta x$ from the point of discharge and then for each time step $(t + \Delta t)$ from the time of release until essentially all the pollutant has been removed from the estuary. Thus a large number of calculations are required for this analysis and computers are usually used in the process. It is important to note, however, that the concentration at any point for the next time step $(c_{x,t+\Delta t})$ is dependent on dispersion out of that segment during this time step and on previous concentration at both the adjacent upstream and downstream segments. This approach, then, recognizes both upstream and downstream advection and dispersion.

6.5.10.2 Non-Conservative Single Discharge Pollutants

As shown on Figure VI-22 this category of discharge includes perhaps the most important of the single event pollutants - storm drain BOD loading. In addition to being dependent on flushing, dispersion time and length of discharge, this group of pollutants is also dependent on decay rates. It is felt that too many approximations and assumptions are involved here for reliable application of hand calculations. Computer models can test a wide range of possible coefficient values and then be calibrated to accurately reflect conditions in a given estuary. Without this "tuning" process the potential error is substantially larger than the predicted concentration values.

It is possible, however, to apply a first order decay term to Equation VI-34 at each time and spatial step to arrive at an equation of the form:

$$c_{x,t+\Delta t} = F\,(c,A,D',t,x)\,e^{-k(t+\Delta t)} \qquad \text{(VI-35)}$$

where

k = first order decay coefficient

$e^{-k(t+\Delta t)}$ = decay term for the time step from t to t+Δt

and

$F(c,A,D',t,x)$ represents the three terms on the right hand side of Equation VI-34.

If dispersion and decay coefficients for a given pollutant in a specific estuary are known over the full length of the estuary, and if good estimates are available for probable initial pollutant distribution in the estuary (preferably through actual field measurement of a previous event), then Equation VI-35 should be applied. Under these conditions it will give acceptable results.

A final suggestion is in order for the analysis of single event discharges. Mathematical results for all of these discharges are dependent on empirical coefficients. Accuracy and reliability will be greatly enhanced, and the guesswork of selecting coefficient values reduced, if they are measured for each estuary. Dye tracers can be injected into an estuary and tracked over time. Surface and bottom floats can be spread over a cross-section at various stages of the tide and traced over time. A wide variety of experimental low-cost techniques are available. The information they provide will not only be the basis for input data to mathematical models but will give the planner an intuitive feeling for the controlling processes in each estuary which will allow him to intelligently interpret and analyze the results obtained from a model.

6.6 THERMAL POLLUTION

6.6.1 General

The presence of one or more major heat sources can have a significant impact on both the local biotic community and local water quality. As a result, consideration of significant thermal discharges by the planner is essential in any comprehensive water quality analysis. Thermal power plants account for the vast majority of both the number of thermal discharges and the total thermal load. However, some industrial process generate significant amounts of excess heat. This discussion will not attempt to address total heat budget calculations. This should be left to computer modeling.

The most important of the impacts of heat discharge are:

1. Ecological Effects: Water temperature increases change the productivity levels for planktonic and many benthic species. As a result local community structures are altered. By itself, this is neither good nor bad. However, many of the species benefited by warmer conditions (e.g., blue green algae) may be considered to be undesirable. In addition, many species can perform certain life cycle functions only within a limited temperature range. Elevated temperatures may prevent some species from completing one or more life stages, thus disrupting the reproductive cycle and destroying the stability of the population.

2. Water Quality Effects: Figure VI-5 showed the relative effect of salinity and ambient temperature on oxygen saturation. From this figure, note the impact on the oxygen saturation concentration of a $10^{o}C$* rise in temperature. This impact is

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*Such a rise is common near power plant thermal plumes.

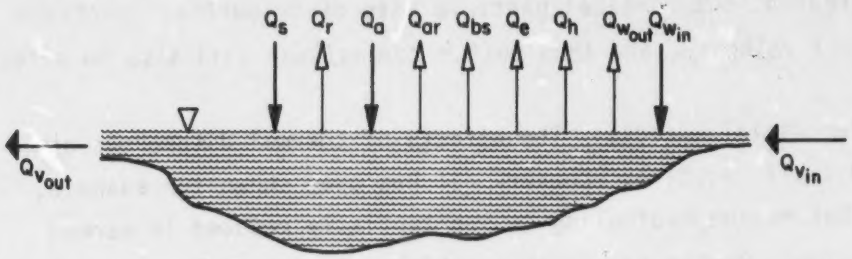
especially significant when ambient temperatures are in the $10^{o}C$ to $15^{o}C$ range which is typical of many estuaries.

3 Sediment Effects: Estuarine sedimentation rates will be increased by increasing local water column temperature. The significance of this increase was discussed by Parker and Krenkel (1970). They concluded that not only will sedimentation rates be increased, but vertical particle size distribution, particle fall velocity, and thus bottom composition will also be affected.

4. Beneficial Effects: The effects of thermal discharges are not all negative, however. It has been shown for example, that marine biofouling is substantially reduced in warmed waters (Parker and Krenkel, 1970). In fact, the recirculation of heated discharge through the condensor has proven to be a less expensive and equally effective method of biofouling control than chlorination for several California coastal power plants. Estuarine contact recreation potentials are increased by increasing local water temperatures, and extreme northern estuaries will have reduced winter ice coverage as a result of thermal discharges.

6.6.2 Approach

A schematic of the heat budget for a segment of an estuary is shown in Figure VI-33. These heat transfer processes (Q_s, Q_a, Q_r, Q_{ar}, Q_{win}) are a function of water temperature, cloud cover, time of day, wind velocity, air temperature, relative humidity, water density and atmospheric pressure. These can all be easily incorporated into a computer model but are awkward for hand calculation. However, the prime concern at this level of analysis is to assess changes in estuarine temperature resulting from man induced changes in thermal loading (e.g., thermal discharges). Therefore, the advective transport of heat through the estuarine water column can be addressed without accounting for heat transport across the water surface. The immediate loss of heat from a thermal plume across the water surface is small and an analysis of advective transport alone will not result in major errors.



$$Q = Q_s - Q_r + Q_a - Q_{ar} - Q_{bs} - Q_e - Q_h - Q_{wont} + Q_{win} + Q_{vin} + Q_{vout}$$

where

Q_s = short wave incident radiation

Q_r = short wave reflected radiation

Q_a = incident long wave radiation

Q_{ar} = reflected long wave radiation

Q_{bs} = long wave back radiation

Q_e = energy utilized in evaporation

Q_h = sensible heat loss

Q_{wout} = heat carried away with evaporated water

Q_{win} = heat bought into estuary with precipitation

Q_{vin} = net advective heat inflow

Q_{vout} = net advective heat outflow

and

Q = net change in water body stored heat

Figure VI-33 Estuarine Heat Budget (After Parker and Krenkel, 1970)

Thus, the approach used here will be to treat excess temperature as a conservative pollutant and to then measure the resulting excess temperature* distribution. Characteristically, thermal discharges are associated with large volumetric water flow rates. As a result, jet and plume effects are often present in the area of the outfall. Thus an analysis of thermal discharges must be divided into two spatial categories - the area immediately surrounding the discharge point which is influenced by momentum effects, and the surrounding nearby area. Consideration of distant points will not be made. If the effects of a thermal discharge are measurably very far away from the discharge point (several miles away, for example) then the water quality effects immediately surrounding the outfall will be extreme. Therefore, prediction of the close-in and intermediate area thermal distribution will suffice to anticipate whether or not a thermal discharge will have important water quality impacts.

Finally, it will be assumed that all discharges are continuous, and only steady-state (non-tidal) conditions will be considered.

6.6.3 Initial Temperature Distribution

The temperature distribution in the immediate vicinity of a thermal discharge is dependent on the discharge structure design. The total advective and dispersive fields transporting heat in this region are determined by the momentum and buoyancy of the discharge

Thermal discharges often have high momentum because of their high velocity and flow rate. The high velocity discharge typically entrains large volumes of ambient water. This will lower the average excess temperature of the water. Entrainment is limited by the ability of ambient water to move into the discharge region and by the spatial expansion patter of the discharge.

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*Excess temperature is a measure of the temperature of a water column above levels measured without the effects of the thermal discharge.

Momentum jet relationships are applicable to the calculation of the thermal distribution in the immediate area of the outfall based on the assumptions and limitations made above. Using dynamic similarity it can be shown that for a jet expanding in two directions (laterally from its center line and downstream) the characteristic velocity will decrease as the square root of distance along the axis, and that for a jet expanding in three directions (laterally and vertically and downstream) the characteristic velocity will decrease linearly with distance (Edinger, 1971).

From this, the following scaling factors between discharge conditions and points downstream may be established for the two dimensional case so that downstream conditions may be analyzed.

1. Momentum conservation:

$$Q_{2x} U_{2x} = Q_o U_o \qquad \text{(VI-36)}$$

2. Dynamic similarity:

$$\frac{U_{2x}}{U_o} = \left(\frac{x_o}{x} \right)^{1/2} \qquad \text{(VI-37)}$$

and 3. (from 1. and 2.):

$$\frac{Q_{2x}}{Q_o} = \left(\frac{x}{x_o} \right)^{1/2} \qquad \text{(VI-38)}$$

where

x = any downstream point measured along plume center line from the point of virtual discharge

x_o = virtual discharge length (similarly Q_o and U_o are the flow rate and velocity at the virtual discharge location

Q_o = plume volumetric flow rate,

U = plume flow velocity

and Q_{2x} and U_{2x} denote 2 dimensional (downstream and lateral) flow rates and velocities

The virtual discharge position (x_o) is that distance downstream from the actual discharge point beyond which discharge jet flow characteristics measurably change from the in-pipe conditions. This distance is generally two to ten times the discharge diameter (Edinger, 1971, and Brooks, 1972). For discharges into flow fields generally perpendicular to the discharge velocity (as is typical for river and estuarine discharge) nearer the lower limit (two times the diameter) is a better estimate of x_o.

It is important to note in Equation VI-38 that the flow dilution ratio (Q_{2x}/Q_o) is independent of discharge characteristics for any given distance downstream. Thus, if Q_o is increased by increasing U_o, entrainment processes will increase proportionately so that Q_{2x}/Q_o will remain unchanged.

Excess temperature may be tracked through the initial mixing stage by assuming excess heat is conserved over the time and space of initial mixing. With this assumption,

$$H \equiv \text{constant} = Q_o \, \theta_o = Q_{2x} \, \bar{\theta}_x \qquad \text{(VI-39)}$$

or,

$$\bar{\theta}_x = \theta_o \frac{Q_o}{Q_{2x}} \qquad \text{(VI-39a)}$$

where H = excess quantity of heat discharged at the outfall per unit of time (same time units as used in "Q" values), and θ = excess temperature at the discharge

For application of these equations, Q_{2x} is calculated from equation VI-38 for some downstream point "x." Then equation VI-39 is used to calculate $\bar{\theta}_x$ for that downstream cross section. It must be remembered that $\bar{\theta}_x$ is averaged over the entire width of each cross section and thus does not represent θ_x for each location across the plume cross section since there is substantial integrity in the central portion of the plume during initial mixing. This is made more evident in Figure VI-34.

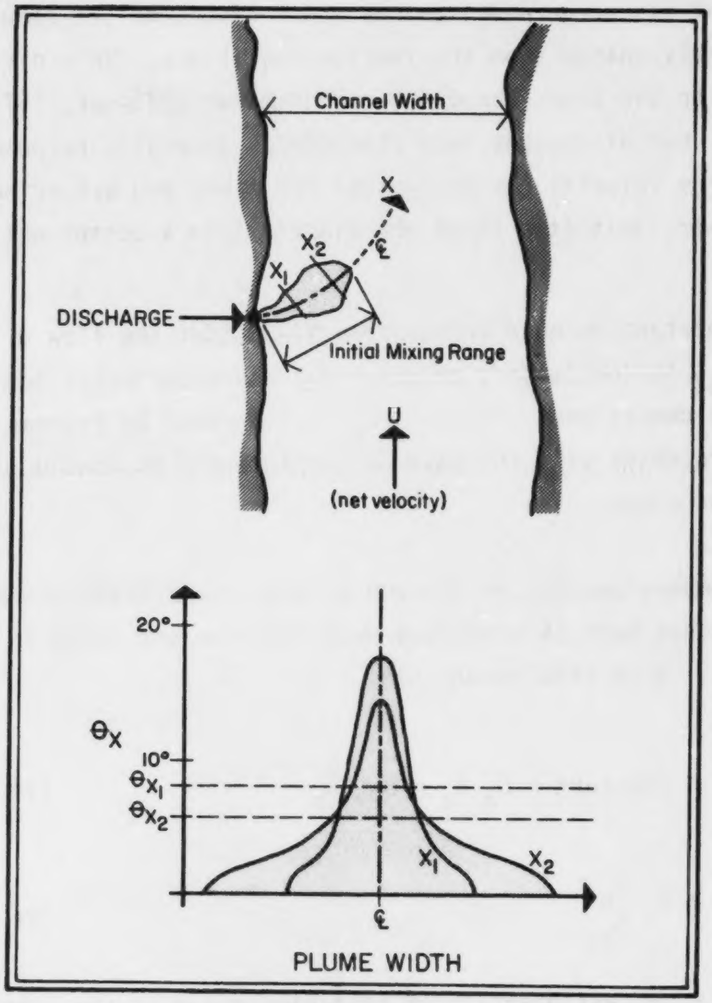
Channel Width
X
X2
℄
X1
DISCHARGE
Initial Mixing Range
U
(net velocity)
20°
θX
10°
θX1
θX2
X1
X2
℄
PLUME WIDTH

FIGURE VI-34 INITIAL THERMAL PLUME MIXING

Within the following limitations, however, equations VI-36 through VI-39 will provide a meaningful picture of plume mixing during this initial momentum mixing process.

1. The equations are only applicable in the area dominated by momentum mixing. While no exact criteria are given to define this region, it is restricted to the immediate downstream area from the jet and typically extends on the order of several hundred feet downstream.

2. The angle of discharge to the estuarine velocity field has been ignored. While changes in the angle of discharge will change x_o values and will change the reach over which these equations are valid, it will not invalidate the use of equations VI-36 through VI-39.

3. Tidal action has been ignored. The system's response to a jet discharge will differ substantially during different stages of the tide. However, this variance will not be excessive in the immediate vicinity of the discharge.

4. Only the two dimensional case has been described. Thus complete initial vertical mixing is assumed so that only lateral and longitudinal plume spread need be addressed. In three dimensional form, equation VI-38 is (Edinger, 1971):

$$\frac{Q_{3x}}{Q_o} = \frac{x}{x_o}$$

   In this case, $\bar{\theta}_x$ is still calculated using equation VI-39.

6.6.4 Intermediate Temperature Distribution

Temperature distributions outside the region of momentum mixing are governed by the advective and dispersive characteristics of the receiving waterbody. The temperature distribution in the intermediate region can be estimated no more precisely than the detail with which velocity and dispersion can be specified for inclusion in a heat conservation equation. The velocity field is no longer controlled by the discharge jet and must be derived from either dynamic equations of motion or from some net non-tidal calculations. Relative to the intermediate temperature distributions, the initial temperature distribution determined by momentum mixing acts like an extension of a thermal plant condenser. The thermal distribution resulting from initial mixing, either by surface or submerged discharges, gives the source thermal distribution to be used as an initial condition for intermediate heat distribution calculation (Edinger, 1971).

Previous simplifications prevent complete analysis of this intermediate temperature distribution. However, Edinger (1971) describes the derivation of a relationship for estimating the area within a given excess heat contour for the two dimensional discharge flow. This relationship is defined as:

$$\frac{A_\theta}{A_{n2}} = 0.168\left(\frac{\theta}{\theta_p}\right)^{-3} \qquad \text{(VI-40)}$$

where: A_θ = surface area with excess temperature $\geq\theta$ (area within a contour line defined by excess temperature = θ)

θ = the excess temperature to be used as a basis for the calculation

θ_p = initial excess temperature of plume,

and A_{n2} = a scaling area (in ft^2) defined by:

$$A_{n2} = 0.715 \frac{Q_p^{\,3}}{E\,U^2\,d^3} \qquad \text{(VI-41)}$$

where Q_p = plume volumetric flow rate after momentum mixing (generally in cfs units)

E = local dispersion coefficient (in ft^2/sec)

U = net non-tidal current velocity (in ft/sec)

and d = average water depth of the plume over the area of intermediate mixing (in ft).

Typically, intermediate mixing extends several thousand feet downstream (Edinger, 1971) so that water depth should be averaged over this distance. Discharge configuration design will determine both whether or not momentum mixing will occur and, if so, over what distance it will dominate the mixing processes. In the absence of any other information, momentum mixing may be assumed to occur over 100 feet to 200 feet. These estimates may then be used as inputs to equations VI-40 and VI-41.

Edinger and Polk (1969) used these relationships to investigate the assumption that excess heat would be conservative over these initial and intermediate areas. They found that inclusion of the various surface heat transport processes had little effect on the resulting areas for excess temperature greater than $0.2\ \theta_p$. Thus equation VI-40 may be used to estimate A_θ for $\theta \geq .2\theta_p$. This A_θ is then the limit of the capability of these calculations to reliably predict an A_θ and this θ is the smallest θ for which these hand calculations should be used. This θ will typically be on the order of 2°F to 4°F.

It must be remembered that there are a large number of approximations and assumptions built into equations VI-40 and VI-41 (E, x_o, x, etc). Thus the resulting areas will also be approximations and are best suited to be "order of magnitude" guides toward the effects of a thermal discharge.

EXAMPLE VI-8

Initial and Intermediate Thermal Mixing

A thermal power plant will be located on the upper reaches of an estuary. Here the net non-tidal flow velocity is 0.6 ft/sec under low river outflow conditions and MLLW depth is 10 feet in the region of the plant. Well-mixed conditions are assumed during this season for this part of the estuary. The plant will discharge 1100 ft^3/sec through a 10 ft. diameter pipe and will have a plant ΔT of 20^oF at rated capacity operation. From work associated with previous sections of this chapter the dispersion coefficient (E) in this region has been estimated to be 100 ft^2/sec*. You want to know what area will be subjected to an excess temperature of 5^oF or more.

According to equation VI-38.

$$\frac{Q_{2x}}{Q_o} = \left[\frac{x}{x_o}\right]^{1/2}$$

Arbitrarily setting $x \approx 200$ ft as the limit of initial mixing, and assuming that $x_o \approx 6$ multiplied by Pipe Diameter,

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*This value is compatable with equation VI-29a and may be typical for the main channel portion of many estuaries.

$$\frac{Q_{2x}}{1100\ \text{ft}^3/\text{sec}} = \left[\frac{200\ \text{ft}}{6\text{x}10\ \text{ft}}\right]^{1/2}$$

or

$$Q_{2x} = 2008\ \text{ft}^3/\text{sec}$$

From equation VI-41,

$$\text{with} \quad Q_p = Q_{2x} = 2008\ \text{ft}^3/\text{sec}$$

$$A_n = 0.751\left(\frac{Q_p{}^3}{E\ U^2\ d^3}\right)$$

$$= 0.751\left(\frac{(2008\ \text{ft}^3/\text{sec})^3}{(100\ \text{ft}^2/\text{sec.})\ (0.6\ \text{ft/sec})^2\ (10\ \text{ft})^3}\right),$$

or $\quad A_n = 168{,}900\ \text{ft}^2$

Then for equation VI-40,

$$A_{(\theta=5^oF)} = 0.168\ A_n\left(\frac{\theta}{\theta_p}\right)^{-3}$$

$$= (0.168)\,(168{,}900\ \text{ft}^2)\ \left(\frac{5^oF}{20^o}\right)^{-3}$$

Thus,

$$A_{\theta=5^oF} = 1.816\ \text{x}\ 10^6\text{ft}^2 = 41.7\ \text{acres}$$

END OF EXAMPLE VI-8

Conditions outside this area of intermediate mixing may be inferred from the outer fringes of the intermediate mixing area. In the above example, $0.2\theta = 4^oF$ and $A_{\theta\ =\ 4^oF}$ = 81.4 acres, or twice the area associated with an excess temperature of 5^oF. It is unlikely that a plume of this surface area would extend much more than 2 to 3 miles downstream. If the resulting thermal regime at that point is acceptable, then points farther downstream in the estuary should also be acceptable as they will be closer to ambient conditions. If thermal conditions farther downstream must be analyzed for some special situation the use of computer models is recommended.

If an estuary is either stratified or partially stratified this procedure may still be used. However, the estuarine depth term (d) in Equation VI-41 should be altered to reflect the depth of the surface water layer providing that the discharge is not made below this surface layer. If the discharge is made into bottom waters, computer or physical models may be required.

6.7 TURBIDITY AND SEDIMENTATION

Turbidity and sedimentation rates are accorded great importance by many decision makers and by much of the general public. Turbidity affects water clarity and water color and hence is of esthetic significance. It also affects light penetration, so that increased turbidity results in a decreased photic zone depth and an associated decrease in primary production. Sedimentation is also of direct economic importance. Dredging and dredge spoil disposal costs represent a large portion of many local and federal water projects. This is especially true for required maintenance dredging in high sedimentation areas. Unfortunately, current state-of-the-art ability to predict sedimentation patterns throughout an estuary, or to predict actual estuarine turbidity levels, is very limited. It is possible, however, to make some general predictions of sediment patterns and to develop planning guidelines for estuarine management with respect to turbidity and sedimentation. Therefore, each topic will be separately addressed here.

6.7.1 Turbidity

Turbidity is a measure of the optical clarity of water and is dependent upon the light scattering and absorption characteristics of both suspended and dissolved material in the water column (Austin, 1974). The physical definition of turbidity is not yet fully agreed upon, and varies from being equal to the scattering coefficient (Beyer, 1969), to the product of an extinction coefficient and measured pathlength (Hodkinson, 1966), and to the sum of scattering and absorption coefficients (VandeHulst, 1957).

Turbidity levels in an estuary are likely to vary substantially in both temporal and spatial dimensions. Temporal variations occur as a function of seasonal river discharge, seasonal water temperature changes, instantaneous tidal current, and wind speed and direction.

Spatially, turbidity varies as a function of water depth, distance from the head of the estuary, water column biomass content, and salinity level. Part of the reason for the wide range of parameters which can affect turbidity levels is that turbidity is the result of both scattering and absorption from both dissolved and suspended material which can be either organic or inorganic in nature. Much of the complexity in the analysis of turbidity results from these different sources of turbidity responding differently to the controlling variables mentioned above. As an example, increased river discharge will tend to increase turbidity because of increased inorganic suspended sediment load. However, such an increase will decrease light penetration, thus reducing water column photosynthesis. This, in turn, will reduce the biologically induced turbidity. In general, suspended sediment loads of silts and clays are dependent upon river discharge. To a lesser extent they result from resuspension of estuarine sediment and from flocculation of very fine colloidal material.

The historical methods employed to monitor turbidity are by the use of a "turbidimeter" and especially by the use of a Jackson candle turbidimeter (Austin, 1974). Light extinction measurements using this instrument are in Jackson Turbidity Units (JTU) which are based on a clay suspension. Once standardized, this arbitrary scale* can be used as a calibration basis to measure changes in turbidity. The calibration JTU scale is shown in Table VI-11. From a measured change in turbidity a relative change in water quality may be inferred. Estuarine water is almost always extremely turbid, especially when compared to ocean, lake, or river waters.

The JTU scale is not the only available turbidity scale. In 1926 Kingsbury and Clark devised a scale based on a Formazin suspension medium which resulted in Formazin Turbidity Units (FTU's). More recently volume scattering functions (VSF) and volume attenuation coefficients (α) have been proposed (Austin, 1974). However, JTU's are still most commonly used as an indicator of estuarine turbidity levels.

As a rough indication of the wide variations possible in turbidity, Figure VI-35 shows suspended solid concentrations for the various subbays of San Francisco Bay for one year (Pearson, et al, 1967). The solid line shows station annual mean concentrations while the dashed lines indicate concentrations exceeded by 20% and 80% of the samples taken at each station over the one year time period. These variations at stations located near bay heads (left and right extremities of Figure VI-35) typically exceed 300% of the annual 20th percentile values. Use of extreme high/low values would produce correspondingly larger annual variations.

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*The JTU scale is an arbitrary scale since it cannot be directly related to physical units when used as a calibration basis for turbidimeter measurement.

TABLE VI-11

GRADIATION OF CANDLE TURBIDIMETER*

Light Path* cm	Turbidity Units	Light Path** cm	Turbidity Units
2.3	1000	11.4	190
2.6	900	12.0	180
2.9	800	12.7	170
3.2	700	13.5	160
3.5	650	14.4	150
3.8	600	15.4	140
4.1	550	16.6	130
4.5	500	18.0	120
4.9	450	19.6	110
5.5	400	21.5	100
5.6	390	22.6	95
5.8	380	23.8	90
5.9	370	25.1	85
6.1	360	26.5	80
6.3	350	28.1	75
6.4	340	29.8	70
6.6	330	31.8	65
6.8	320	34.1	60
7.0	310	36.7	55
7.3	300	39.8	50
7.5	290	43.5	45
7.8	280	48.1	40
8.1	270	54.0	35
8.4	260	61.8	30
8.7	250	72.9	25
9.1	240		
9.5	230		
9.9	220		
10.3	210		
10.8	200		

**Measured from inside bottom of glass tube.
*from Austin, 1969, pg. 38

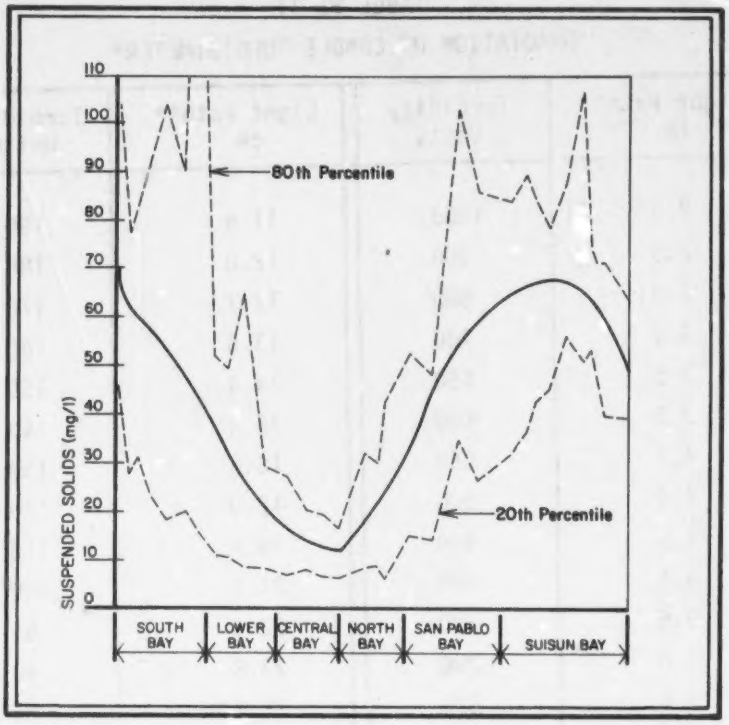

FIGURE VI-35 MEAN SUSPENDED SOLIDS IN SAN FRANCISCO BAY
FROM: PEARSON *ET AL.*, 1967, PG V-15

6.7.2 Sedimentation

Like turbidity, sedimentation is a multifaceted phenomenon in estuaries. As in rivers, estuaries transport bed load and suspended sediment. However with the time varying currents in estuaries, no equilibrium or steady state conditions can be achieved. (Ippen, 1966). Additionally, while any given reach of a river will have reasonably constant water quality conditions, an estuary varies from fresh water (<1 p.p.t. salinity) to sea water (≥ 30 p.p.t. salinity), and from a normally slightly acidic condition near the head to a slightly basic at the mouth. The responses of many dissolved and suspended sediments vary substantially across these pH and salinity gradients. Many colloidal sized particles* agglomerate and settle to the bottom.

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*Colloidal particles are particles small enough to remain suspended by the random thermal motion of the water.

In general, all estuaries undergo active sedimentation which tends to fill in the estuaries (U.S. Engineering District, San Francisco, 1975). It is also true for essentially all U.S. estuaries that the rate of accumulation of sediment is limited not by the available sources of sediment but by the estuary's ability to scour unconsolidated sediments from the channel floor and banks.

6.7.2.1 Qualitative Description of Sedimentation

Before presenting what quantitative information is available concerning sediment distribution in an estuary, a qualitative description of sediment sources, types and distribution will be helpful. Sediment sources may be divided into two general classes: sources external to the estuary and sources internal to the estuary (Schultz and Simmons, 1957). The major sources of sediment within each category are shown below. By far the largest external contributor is the upstream watershed.

1. External:

   - Upstream watershed
   - Banks and stream bed of tributaries
   - Ocean areas adjacent to the mouth of the estuary
   - Surface runoff
   - Wind borne sediments
   - Human input (M & I discharges)

2. Internal

   - Estuarine marsh areas
   - Wave and current resuspension of unconsolidated bed materials
   - Flocculation
   - Estuarine biological activity
   - Human input (dredging)

General characterizations of U.S. estuarine sediments have been made by Ippen (1966) and by Schultz and Simmons (1957). Many individual case study reports are available for sediment characterization of most of the larger U.S. estuaries (i.e. Columbia River, San Francisco Bay, Charleston Harbor, Galveston Bay, Savannah Harbor, New York Harbor, Delaware River and Bay, etc.). In general, estuarine sediments range from fine granular sand (0.01 in. to 0.002 in. in diameter) through silts and clays to fine colloidal clay (0.0003 in. or less in diameter) (Ippen, 1966). Very little, if any, larger material (coarse sand, gravel, etc.) is found in estuarine sediments. Sand plays a relatively minor role in East Coast, Gulf Coast and Southern Pacific Coast estuaries. Usually it constitutes less than 5% by volume (< 25% by weight) of total sediments for these estuaries with most of this sand concentrated near the estuarine mouth (Schultz & Simmons, 1957). By contrast, sand is a major element in estuarine shoaling for the north Pacific estuaries (i.e. Washington and Oregon coasts). These estuaries are characterized by extensive oceanic sand intrusion into the lower estuarine segments and by extensive bar formations near the estuarine mouth. The relative distribution of silts and clays of organic and inorganic material within different estuaries, and, in fact, the distribution of shoaling and scour areas within estuaries, varies widely.

6.7.2.2 Estuarine Sediment Forces and Movement

As sediments enter the lower reaches of a river and come under tidal influence they are subjected to a wide variety of forces which control their movement and deposition. First, net velocities in the upper reaches of estuaries are normally lower than river velocities. Additionaly, the water column comes under the influence of tidal action and thus is subject to periods of slack water. During these periods coarse sand and larger materials tend to settle. Once settled, the scour velocity required to resuspend the material is higher than that required to carry a particle when in suspension. Thus, once the coarser particles settle out in the lower river and upper estuarine areas, they tend not to be resuspended and carried farther into the estuary (U.S. Engineering District, San Francisco, 1975). Exceptions to this principle can come during periods of extremely high river discharge when water velocities can hold many of these particles in suspension well into or even through an estuary. Table VI-12 lists approximate maximum allowable velocities to avoid scour for various sizes of exposed particles. Values are approximate and are for unarmored sediment (sediment not protected by a covering of larger material).

Sediment are subject to gravitational forces and have size dependent settling velocities. However, in highly turbulent waters these particle fall velocities are small compared to background fluid motion. Thus the influence of natural gravitational forces on sediment distribution is primarily limited to the relatively quiescent, shallow areas of estuaries and to periods of slack water. Some information can be obtained, however, by comparing residence time and size dependent settling velocities to known sedimentation patterns. This will be covered further in subsequent paragraphs in this section.

As mentioned earlier, particle settling attains a maximum in each tidal cycle during high water slack and low water slack tides. During periods of peak tidal velocity (approximately half way between high and low water) resuspension of unconsolidated sediment may occur. Thus during a tidal cycle large volumes of sediment are resuspended, carried upstream with flood flow, deposited, resuspended, and carried downstream on the ebb tide. Only those particles deposited in relatively quiescent areas have the potential for long term residence. Compounding this cyclic movement of sediments are seasonal river discharge variations which alter estuarine hydrodynamics. Thus, sediment masses tend to shift from one part of an estuary to another (Schultz and Simmons, 1975).

As fresh waters encounter areas of significant salinity gradients extremely fine particles (primarily colloidal clay minerals) often destabilize (coagulate) and agglomerate to form larger particles (flocculate). The resulting floc (larger agglomerated masses) then settle to the bottom. Coagulation occurs when electrolytes, such as magnesuim sulfate and sodium chloride, "neutralize" the repulsive forces between clay particles. This allows the particles to adhere upon collision (flocculation), thus producing larger masses of material. Flocculation rates are dependent on the size distribution and relative composition of the clays and electrolytes and upon local boundary shear forces (Ippen, 1966, and Schultz and Simmons, 1957). Flocculation occurs primarily in the upper central segments of an estuary in the areas of rapid salinity increase.

Movement of sediments along the bottom of an estuary does not continue in a net downstream direction as it does in the upper layers and in stream reaches. In all but a very few extremely well mixed estuaries upstream bottom currents predominate at the mouth of an estuary. Thus there is a greater upstream flow at the bottom than downstream flow. This is counterbalanced by increased surface downstream flow. However, net upstream flow along the bottom results in a net upstream transport of sediment along the bottom of an estuary near the mouth. Thus, sediments and flocs settling into the bottom layers of an estuary near the mouth are often carried back into the estuary rather than being carried out into the open sea. Consequently, estuaries tend to trap, or to conserve sediments while allowing fresh water flows to continue on out to sea (Strumm and Morgan, 1970). At some point along the bottom, the upstream transport will be counterbalanced by the downstream transport from the fresh water inflow. At this point, termed the "null zone," there is essentially no net bottom transport. Here sediment deposition is extensive. In a stratified estuary this point is at the head of the saline intrusion wedge. In a partially well mixed estuary it is harder to pinpoint. Nonetheless, sedimentation is a useful parameter to analyze and will be handled in a quantitative manner in subsequent paragraphs (Section 6.7.2.3.).

TABLE VI-12

MAXIMUM ALLOWABLE CHANNEL VELOCITY TO AVOID BED SCOUR (FPS) (KING, 1954)

Original material excavated	Clear water, no detritus	Water transporting colloidal silts	Water transporting non-colloidal silts, sands, gravels or rock fragments
Fine sand	1.50	2.50	1.50
Sandy loam.	1.75	2.50	2.00
Silt loam	2.00	3.00	2.00
Alluvial silts.	2.00	3.50	2.00
Ordinary firm loam.	2.50	3.50	2.25
Volcanic ash.	2.50	3.50	2.00
Fine gravel	2.50	5.00	3.75
Stiff clay.	3.75	5.00	3.00
Graded, loam to cobbles	3.75	5.00	5.00
Alluvial silt	3.75	5.00	3.00
Graded, silt to cobbles	4.00	5.50	5.00
Coarse gravel	4.00	6.00	6.50
Cobbles and shingles.	5.00	5.50	6.50
Shales and hardpans	6.00	6.00	5.00

To this point, flow in a fairly regular channel has been assumed. However in many estuaries topographical irregularities exist. Such irregularities (e.g., narrow headlands) create eddy currents in the lee of the feature. These eddy currents, or gyres, slow the sediment movement and allow local shoaling. Additionally, large shallow subtidal or tidal flatlands exist in many estuaries. Such areas are usually well out of the influence of primary currents. As a result local water velocities are usually low and increased shoaling is possible.

Wind and waves also have a major influence on estuarine sediment distribution. Seasonal wind driven currents can significantly alter water circulation patterns and associated velocities. This in turn determines, to a large extent, the areas of net shoaling and scour throughout an estuary. Local wind driven and oceanic waves can create significant scour forces. Such scour, or particle resuspension, is particularly evident in shallow areas where significant wave energy is present at the sediment/water interface. Local wind driven waves are a major counterbalancing force to low velocity deposition in many estuarine shallow areas (U.S. Engineering District, San Francisco, 1975).

Finally, oceanic littoral currents (long shore currents) interact with flood and ebb flows in the area of an estuary mouth. Particularly in the Pacific Northwest, sandy sediment fed from such littoral drift is a major source of estuarine sediment, and the interference of littoral drift with normal flood and ebb flows is the major factor creating estuarine bars.

Figure VI-36 shows the schematic flow of annual sediment movement through San Francisco Bay. With the exception of the magnitude of annual dredging, this is typical for most U.S. estuaries. The most important thing to observe is the dominance of resuspension and redeposition over all other elements of sediment movement including net inflow and outflow. Also note that there is a net annual accumulation of deposited sediment in the bay. This figure is also helpful in conceptualizing the sediment trap or sediment concentration characteristic of estuaries. In any year, 8-10 million cubic yards flow into the estuary and 5 to 9 million cubic yards flow out. However, over 180 million cubic yards are actively involved in annual sediment transport within the estuary.

Figure VI-37 is an idealized conceptualization of the various sediment-related processes in an estuary. It must be remembered that

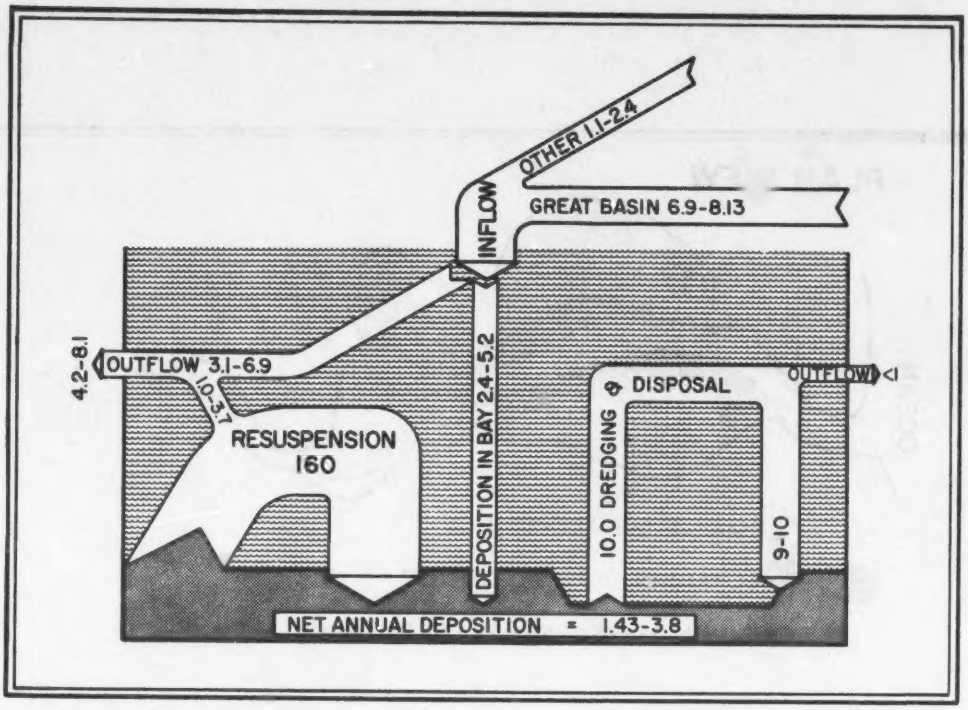
OTHER 1.1-2.4
INFLOW
GREAT BASIN 6.9-8.13
4.2-8.1
OUTFLOW 3.1-6.9
1.0-3.7
RESUSPENSION
160
DEPOSITION IN BAY 2.4-5.2
10.0 DREDGING & DISPOSAL
OUTFLOW <1
9-10
NET ANNUAL DEPOSITION = 1.43-3.8

FIGURE VI-36 SEDIMENT MOVEMENT IN SAN FRANCISCO BAY SYSTEM (MILLION CUBIC YARDS). FROM: U.S. ENGINEERING DISTRICT, SAN FRANCISCO, 1975)

these processes actually overlap spatially much more than is shown and that the processes active at any given location vary considerably over time.

From this qualitative analysis, there are some general statements which can be made. Ippen (1966) drew the following conclusions on the distribution of estuarine sediments:

a) The major portion of sediments introduced into suspension in an estuary from whatever source (includes resuspension) during normal conditions will be retained therein, and if transportable by the existing currents will be deposited near the ends of the salinity intrusion, or at locations of zero net bottom velocity.

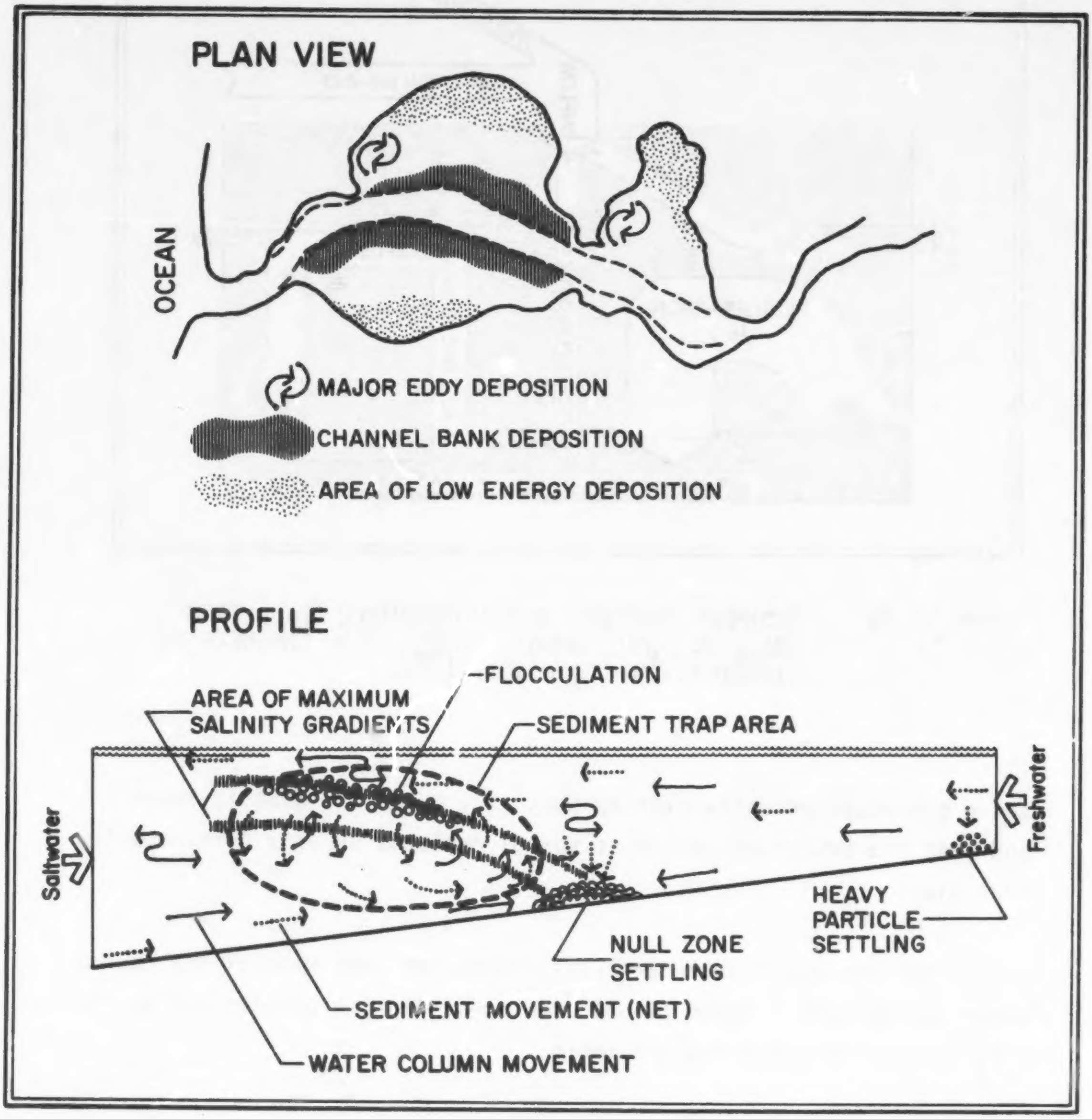
PLAN VIEW
OCEAN
MAJOR EDDY DEPOSITION
CHANNEL BANK DEPOSITION
AREA OF LOW ENERGY DEPOSITION
PROFILE
FLOCCULATION
AREA OF MAXIMUM SALINITY GRADIENTS
SEDIMENT TRAP AREA
Saltwater
Freshwater
HEAVY PARTICLE SETTLING
NULL ZONE SETTLING
SEDIMENT MOVEMENT (NET)
WATER COLUMN MOVEMENT

FIGURE VI-37 IDEALIZED ESTUARINE SEDIMENTATION

b) Any measure contributing to a shift of the regime towards stratification will cause increased shoaling. Such measures may be: structures to reduce the tidal flow and prism, diversion of additional fresh water into the estuary, deepening and narrowing of the channel,

c) Sediments settling to the bottom of an estuary will, on the average, be transported upstream and not downstream. Such sediments may, at some upstream point be resuspended into the upper layers and carried back downstream.

d) Sediments will accumulate near the ends of the intrusion zone and form shoals. Shoals will also form where the net bottom velocity is zero (in the null zone),

e) The intensity of shoaling will be most extreme near the end of the intrusion for stratified estuaries and will be more dispersed in the well mixed estuary,

f) Shoals will occur along the banks of the main estuarine channel where water is deep enough to prevent wave induced scour and where velocities are reduced from main channel velocities sufficiently to allow settling.

Schultz and Simmons (1957) made similar conclusions but added the presence of shoaling at the mouth where flood and ebb currents intercept littoral drift.

6.7.2.3 Settling Velocities

As was stated in the previous section, settling velocities do not play as major a role in controlling sedimentation patterns in estuaries as they do in lakes. However, it is informative to assess settling rates for various size particles. The possible size classifications of particles and their general inclusive diameter sizes are shown in Table VI-13. Table VI-14 lists terminal settling velocities for each particle size assuming spherical particles and density of 2.0* in quiescent water. From this table it can be inferred that particles of the medium sand class and coarser will probably settle to the bottom within a very short time after entering an estuary.

Turning to the other end of the particle size scale of Table VI-14, particles with a diameter of 10^{-4} mm will fall only 3.1×10^{-3} inches per hour in the most favorable environment (calm waters). Such a settling rate is not significant in the estuarine environment. Figure VI-38 shows the quiescent settling rates for particle sizes in between these two extremes since this intermediate size group is of real significance in estuarine management (primarily silts). For particles smaller than those shown in Figure VI-36, gravitational settling will not be a significant factor in controlling particle motion. Particles substantially larger than the range shown in Figure VI-38 will tend to settle above, or at, the head of an estuary.

Combining Figure VI-38 (fall per tidal cycle)** with known segment flushing times (in tidal cycles) the size of particles tending to settle out in each segment can be estimated. If such predictions reasonably match actual mean segment sediment particle size, then this method can be useful in predicting changes in sediment pattern. Anticipated changes in river-borne suspended sediment load by particle size can be compared to areas where each size of particle would tend to settle. This would then identify areas which would either be subject to increased shoaling or reduced shoaling and increased scour. This type of analysis has been more successful when applied to organic detritus material than for inorganic suspended loads.

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*The density of many inorganic suspended particles is approximately equal to that of sand (2.7 gm/cm^3) while that of biomass and organic detritus is usually much closer to that of water and can be assumed to be about 1.1 gm/cm^3.

**Based on a 12.4 hour tidal cycle

TABLE VI-13

SEDIMENT PARTICLE SIZE RANGES (AFTER HOUGH, 1957)

	PARTICLE SIZE RANGE			
	Inches		Millimeters	
	$D_{max.}$	$D_{min.}$	$D_{max.}$	$D_{min.}$
Derrick STONE	120	36	--	--
One-man STONE	12	4	--	--
Clean, fine to coarse GRAVEL	3	1/4	80	10
Fine, uniform GRAVEL	3/8	1/16	8	1.5
Very coarse, clean uniform SAND	1/8	1/32	3	0.8
Uniform, coarse SAND	1/8	1/64	2	0.5
Uniform, medium SAND	--	--	0.5	0.25
Clean, well-graded SAND AND GRAVEL	--	--	10	0.05
Uniform, fine SAND	--	--	0.25	0.05
Well-graded, silty SAND AND GRAVEL	--	--	5	0.01
Silty SAND	--	--	2	0.005
Uniform SILT	--	--	0.05	0.005
Sandy CLAY	--	--	1.0	0.001
Silty CLAY	--	--	0.05	0.001
CLAY (30 to 50% clay sizes)	--	--	0.05	0.0005
Collodal CLAY ($\bar{-}2\mu>50\%$)	--	--	0.01	10^{-6}

(After B. K. Hough, Basic Soils Engineering, p. 69, Values listed are approximate)

A number of simplifying assumptions have gone into this settling velocity analysis. The most significant of these are:

1. Water column density changes have been ignored. Inclusion of this factor would slightly reduce the settling velocity with increased depth. This effect will be more significant for organic matter because of its lower density.

TABLE VI-14

RATE OF FALL IN WATER OF SPHERES OF VARYING RADII AND CONSTANT DENSITY OF 2[a] AS CALCULATED BY STOKES' LAW[b] (MYSELS, 1959)

Radius	Terminal velocity, u	
mm.	cm./sec.	cm./min.
10	(2.2×10^{4})	
1	(220)	
0.1	(2.2)	
0.01	2.2×10^{-2}	1.3
10^{-3}	2.2×10^{-4}	0.013
10^{-4}	2.2×10^{-6}	1.3×10^{-4}
10^{-5}	2.2×10^{-8}	1.3×10^{-6}
10^{-6}	2.2×10^{-10}	1.3×10^{-8}
10^{-7}	(2.2×10^{-12})	

[a] To apply to other conditions, multiply the u value by the pertinent density difference and divide it by the pertinent viscosity in centipoises.

[b] Values in parentheses are calculated by Stokes' law under conditions where this law is not applicable.

[c] Stokes law states that the terminal velocity is proportional to the particle radius squared, the difference in density and inversely proportional to the liquid viscosity.

2. Dispersive phenomena and advective velocities have not been considered.

3. Table VI-13 and Figure VI-38 are based on the fall of perfectly spherical particles. Non-spherical particles will have lower settling velocities.

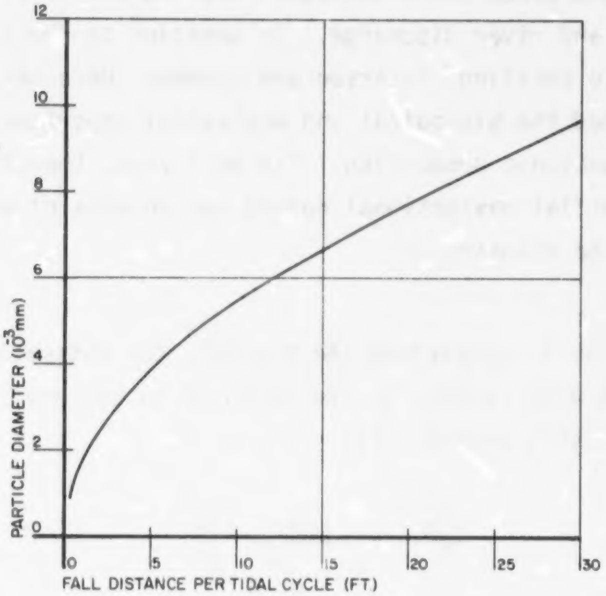


FIGURE VI-38. PARTICLE DIAMETER VS SETTLING FALL PER TIDAL CYCLE (12.4 HRS) UNDER QUIESCENT CONDITIONS (SPHERES WITH DENSITY 2.0 GM/CM^3)

4. Interference between particles has not been considered. However, in a turbulent, sediment-laden estuary such interference is probable (hindered settling). The analysis of the effect of interference on settling velocities was covered in Chapter V for lakes. This analysis is also basically valid for estuaries. The effects introduced there can be applied to Figure VI-38 velocities to adjust for particle interference.

6.7.2.4 Null Zone Calculations

It was previously mentioned that substantial shoaling will occur in the area of the null zone. It is possible to estimate the location of this zone, and hence the associated shoaling areas, as a function of water depth and river discharge. In addition to the importance of the null zone to shoaling, Peterson and Conomos (Peterson, et al., 1974) established the biological and ecological importance of this area in terms of planktonic production. The null zone, therefore, is both an area of potential navigational hazard and an area of major ecological importance to the planner.

Silvester (1974) summarized the analysis for estimating the location of the null zone with respect to the mouth of an estuary. The basic equation used in this analysis is:

$$\frac{\bar{S}_n}{S_o} = \frac{1000}{0.7 S_o F_n^2} \frac{U_r^2}{gd} \qquad \text{(VI-42)}$$

Where $\bar{S}_n$ = mean salinity (averaged vertically and over a tidal cycle) at the null point (n), (ppt)

S_o = ocean surface salinity adjacent to the esturary in parts per thousand (ppt),

U_r = fresh water flow velocity, (ft/sec)

g = gravational force = 32.2 ft/sec.^{2},

d = estuarine depth, (ft)

and F_n = densimetric Froude number at the null zone where F_n is defined by:

$$F_n = U_r / \sqrt{(\Delta\rho/\rho_n) gd} \qquad \text{(VI-43)}$$

where

$\Delta\rho/\rho_n$ = difference between fresh water density and that at the null zone (averaged over the depth of the water column) divided by the density of the null zone. This value may be approximated for estuarine waters by:

Combining Equations VI-42 and VI-44 and solving for $\Delta\rho/\rho_n$ yields

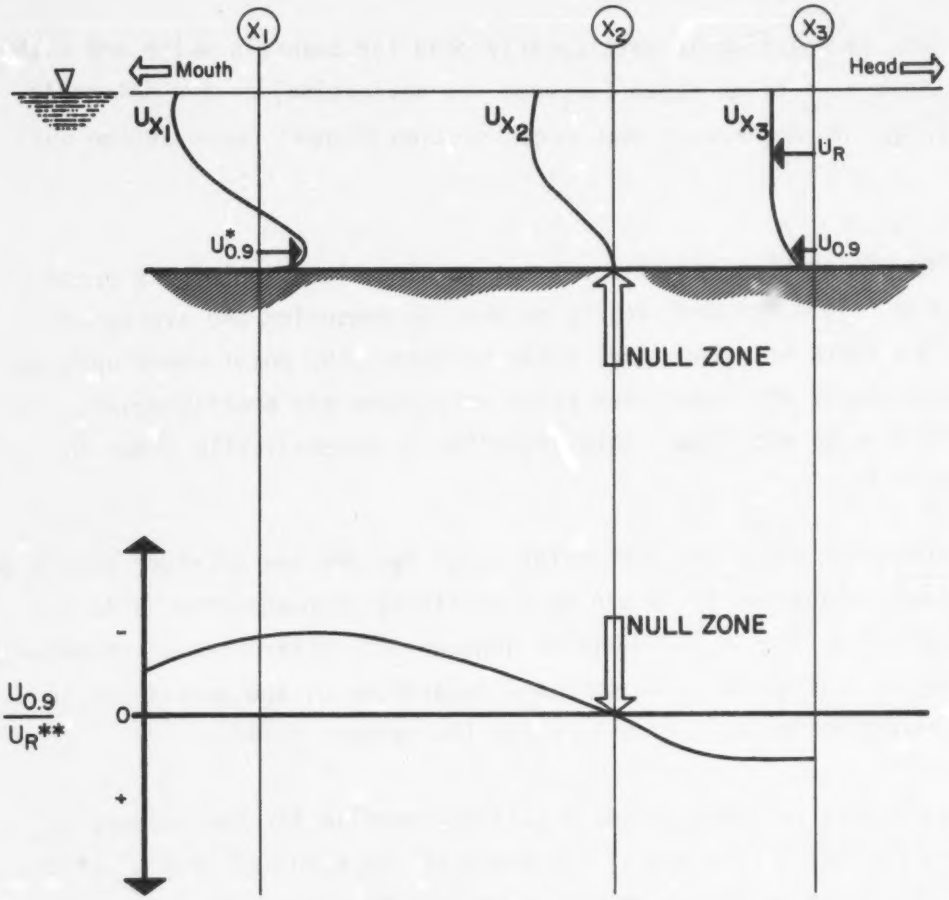
$$\Delta\rho/\rho_n = \frac{0.7}{1000}\ \bar{S}_n \qquad \text{(VI-44)}$$

This formulation is particularly good for channels which are either maintained at a given depth (dredged for navigation) or are naturally regular as "d" represents mean cross section channel depth at the null zone.

The use of these equations first requires location of the present null zone. This can most easily be done by measuring and averaging bottom currents over one tidal cycle to locate the point where upstream bottom currents and downstream river velocities are exactly equal, resulting in no net flow. This situation is schematically shown in Figure VI-39.

When this point has been established for one set of river discharge conditions, equation VI-44 can be substituted into equation VI-43 to calculate F_n. This F_n value is an inherent characteristic of an estuary and can be considered to be constant regardless of the variations in flow conditions or null zone location (Silvester, 1974).

With this information and a salinity profile for the estuary (S_x plotted against x from x=0 at the mouth of the esturary to $X=L_e$ at the head) the location of future null zones may be calculated. Given the new conditions of U_r (changes in river discharge) or of d (changes in channel depth, as by dredging activity), Equation VI-42 will allow calculation of a new $\bar{S}_n$. This may be plotted on the salinity profile to calculate the location of a new null zone position. Even through these changes will produce a new estuarine salinity profile, the use of Equation VI-42 and the old (known) salinity profile will produce reasonably good estimates of longitudinal shifts in the location of the null zone. Salinity profiles for appropriate seasonal conditions should be used for


X1
X2
X3
Mouth
Head
U_{X_1}
U_{X_2}
U_{X_3}
U_R
$U^*_{0.9}$
$U_{0.9}$
NULL ZONE
NULL ZONE
-
$\frac{U_{0.9}}{U_R**}$
0
+

* $U_{0.9}$ = tidally veraged velocity at a depth equal to 0.9 of the water column depth.

** U_R = river flow velocity

FIGURE VI-39 ESTUARINE NULL ZONE IDENTIFICATION

each calculation (e.g. low flow profiles for a new low flow null zone calculation).

EXAMPLE VI-9

Estimation of Null Zone Location

An estuary has the tidally averaged salinity profile shown in the Salinity Table below. Mean channel depth in the area of the existing null zone is 18 feet and the salinity at that point is 10 parts per thousand (ppt). Current (low flow) river discharge velocity is 0.5 ft/sec. Normal winter (high flow) velocity is 1.8 ft/sec. It is desired to know where the null zone will be located in summer and winter if a 30 ft channel is dredged up to 70,000 feet from the mouth.

SALINITY DATA FOR EXAMPLE VI-9

Distance from mouth (1000ft)	5	15	25	35	45	55	65	75	85
Salinity. (ppt)	30	28	25	20	13	8	6	4	1

From equation VI-43 and equation VI-44

$$F_n = U_r \Big/ \sqrt{(.7/1000)\ (\bar{S}_n)\ (g)\ (d)}$$

$$= 0.5\ \text{ft/sec} \Big/ \sqrt{(7\times10^{-4})\ (10\ \text{ppt})\ \ (32.2\ \text{ft/sec}^2)\ (18\ \text{ft})}$$

or, $F_n = 0.248$

From equation VI-42 the null zone salinity with a deeper channel will be:

$$\bar{S}_n = \frac{S_o\ 1000\ U_r^2}{S_o\ 0.7\ F_n^2\ gd}$$

$$= (1000)\ (0.5\ \text{ft/sec})^2 \big/ 0.7\ (0.248)^2\ (32.2\ \text{ft/sec}^3)\ (30\ \text{ft})$$

$$\bar{S}_n = 6.0\ \text{ppt}$$

From Table VI-15 this will occur approximately 65,000 ft from the mouth of the estuary.

Under winter flow conditions,

$$\bar{S}_n = \frac{1000\ \ U_r^2}{0.7\ F_n^2\ gd}$$

$$= (1000)\ (1.8\ \text{ft/sec}) \big/ 0.7\ (0.248)^2\ (32.2\ \text{ft/sec}^2)\ (30\ \text{ft})$$

$$\bar{S}_n - 77.9\ \text{p.p.t.}$$

This $\bar{S}_n$ is greater than ocean salinity and will not actually be encountered. Thus null zone shoaling will occur at the mouth if it occurs at all. This condition is common for rivers with highly seasonal dependent flow rates.

——————— END OF EXAMPLE VI-9 ———————

6.8 TIDAL PRISM-THROAT CROSS SECTIONAL AREA RELATIONSHIPS

A useful planning relationship can be developed between an estuarine tidal prism and the throat cross sectional area. The "throat" of an estuary may be defined as the smallest cross section in the area of the mouth of an estuary. This usually corresponds to the point where the entrance channel is narrowest. The maintenance of this cross sectional area as an open, viable navigational channel is dependent on the volume and velocity of tidal prism water movement through the throat. This relationship has been studied using regression techniques by O'Brien (1969), and Johnson (1973), and more recently by Jarrett (1976).

O'Brien developed a regression expression based on data from all types of estuaries. This relationship is given by:

$$A = 4.69 \times 10^{-4} P^{0.85} \qquad \text{(VI-45)}$$

where, A = throat cross sectional area at mean sea level (ft^2)
P = tidal prism (ft^3)

Johnson analyzed west coast estuaries only, and segregated them by the number of jetties existing at the mouth. His resulting formulation was:

$$A = 2 \times 10^{-5} P \quad \text{for no jetties, } ft^2\text{, } P \text{ in } ft^3 \qquad \text{(VI-46)}$$

and

$$A = 4.69 \times 10^{-4} P \quad \text{for 2 jetties, } ft^2 \; P \text{ in } ft^3 \qquad \text{(VI-47)}$$

Estuaries with one jetty would lie between these two values.

Jarrett made separate analyses for estuaries with no jetties, with one jetty and with two jetties for the Atlantic, Gulf and Pacific coasts. His resulting regression relationships are shown in Table VI-15. Figures VI-40, VI-41, and VI-42 show all of these equations in graphical form.

Knowning the estuarine tidal prism, Figures VI-40, VI-41, and VI-42 may be used to assess the impact on shoaling or scour in the throat area of either jetty construction or of dredging activity (increasing A). If jetties are constructed the "natural" supportable throat area will be difined by a new equation depending on the final number of jetties.

TABLE VI-15

THROAT AREA - TIDAL PRISM RELATIONSHIPS

Coast	A=* All estuaries	A=* 0 or 1 Jetty	A=* 2 Jetties
Atlantic	$7.75\times10^{-6}P^{1.05}$	$5.37\times10^{-6}P^{1.07}$	$5.77\times10^{-5}P^{0.95}$
Gulf	$5.02\times10^{-4}P^{0.84}$	$3.51\times10^{-4}P^{0.86}$	--
Pacific	$1.19\times10^{-4}P^{0.91}$	$1.91\times10^{-6}P^{1.1}$	$5.28\times10^{-4}P^{0.85}$
Combined	$5.74\times10^{-5}P^{0.95}$	$1.04\times10^{-5}P^{1.03}$	$3.76\times10^{-4}P^{0.86}$

(from Jarrett, 1976)

*Values given are representation of right hand sides of equations in the form of Equation VI-45, 46, or Equation VI-47, and are for A in units of ft^2 and for P in units of ft^3.

If throat area dredging is undertaken, the tendency of the estuary to either maintain or to refill this increased area may be directly determined from the appropriate figure (VI-40, 41, or 42).

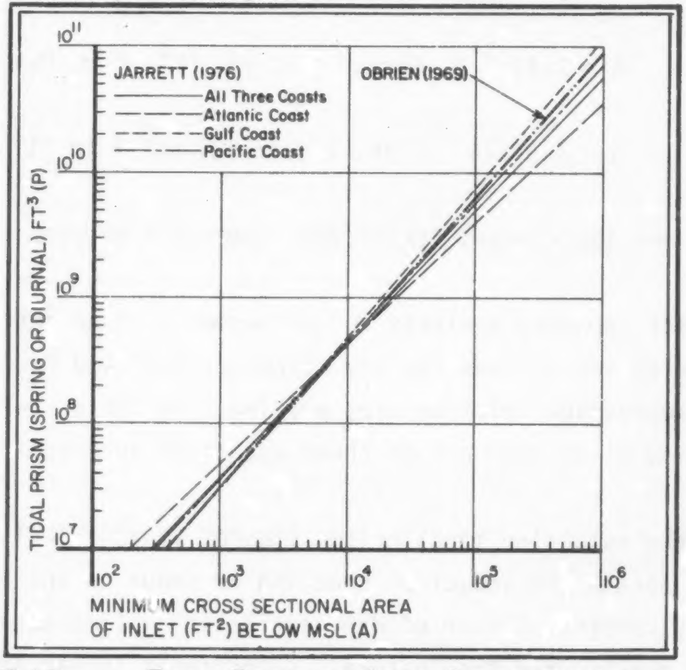


FIGURE VI-40. TIDAL PRISM VS CROSS-SECTIONAL AREA, REGRESSION CURVES FOR ALL INLETS

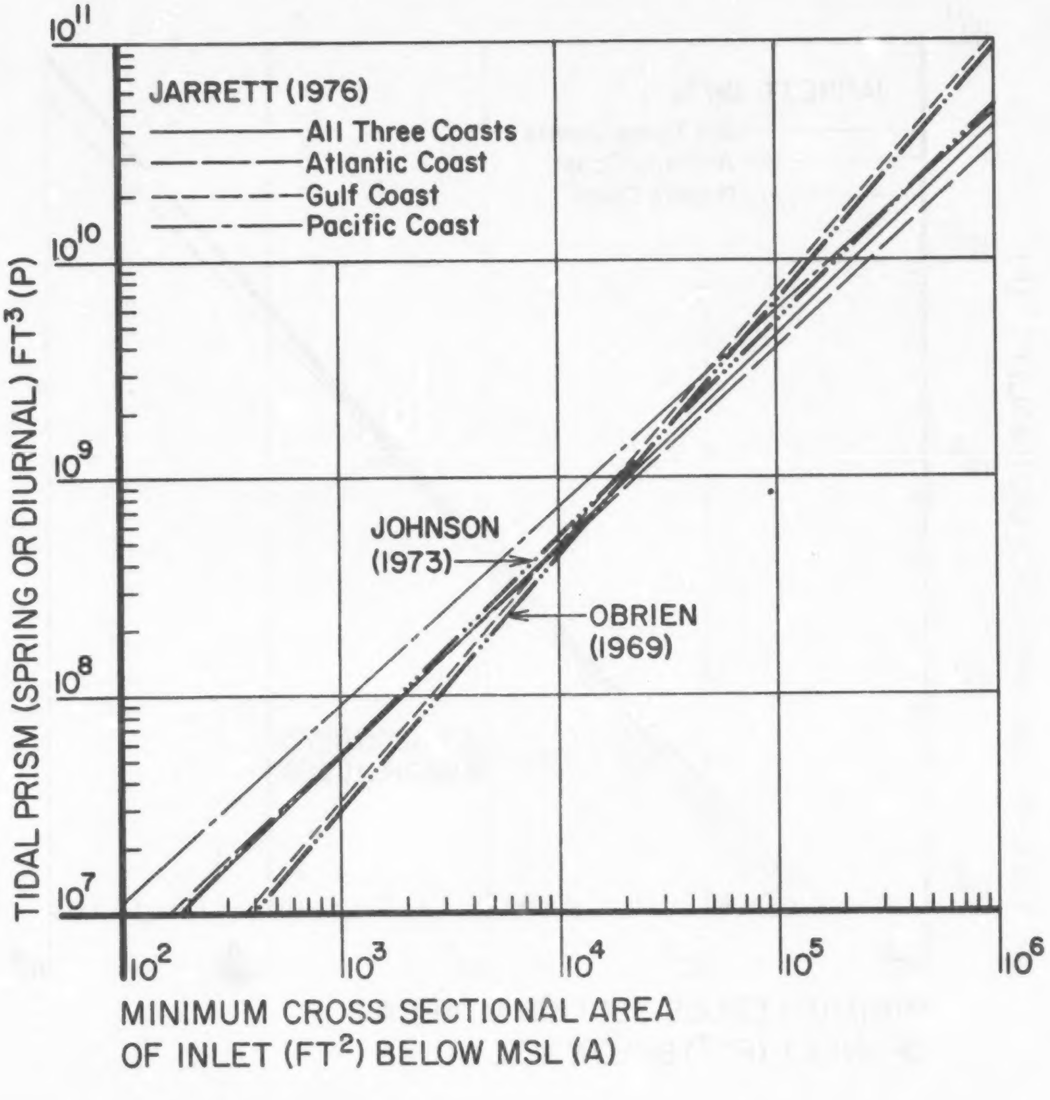


Figure VI-41 Tidal Prism vs Cross-Sectional Area, Regression Curves for Inlets with One or No Jetties

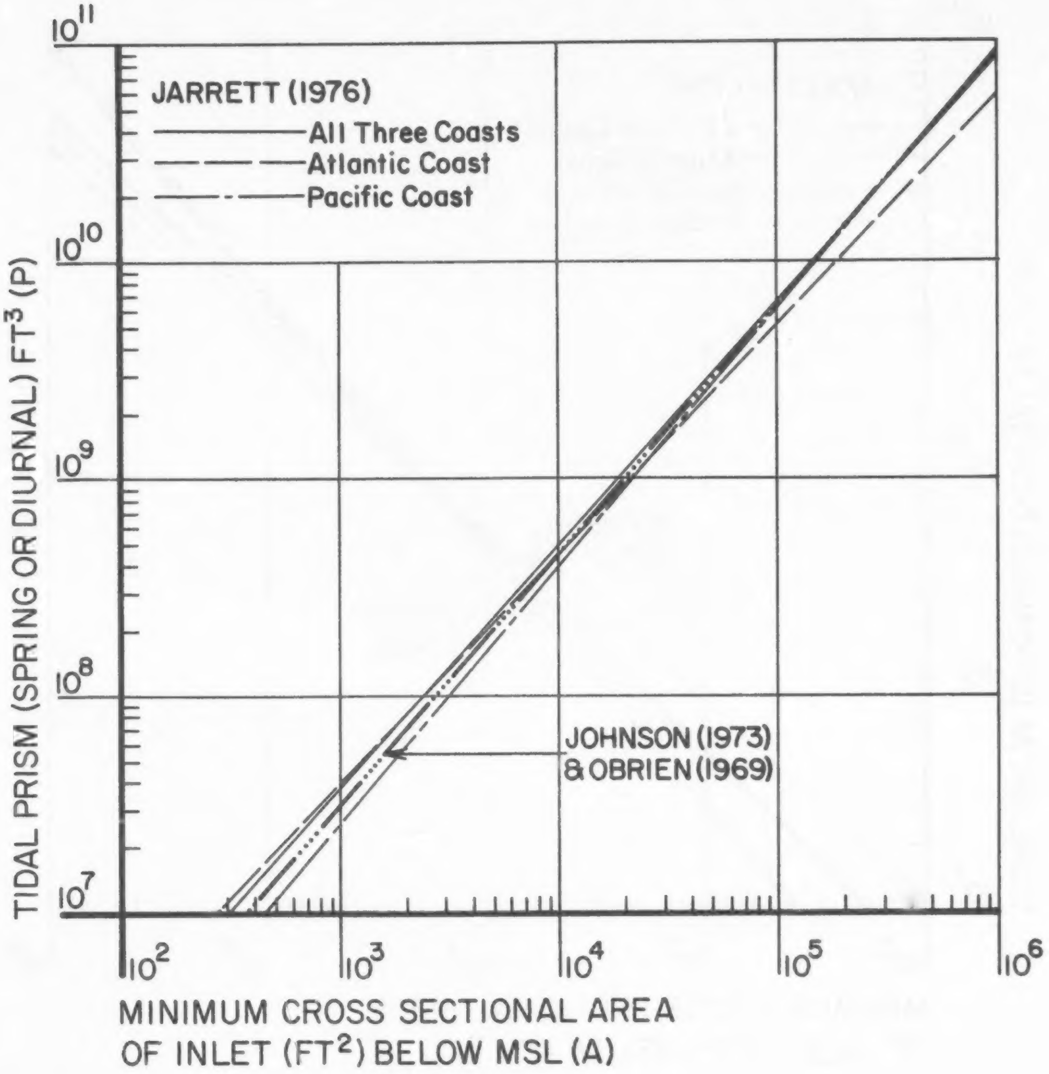


FIGURE VI-42 TIDAL PRISM VS CROSS-SECTIONAL AREA, REGRESSION CURVES FOR INLETS WITH TWO JETTIES

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APPENDIX A

MONTHLY DISTRIBUTION OF RAINFALL EROSIVITY FACTOR R

Figure A-1 - Key Map for Selection of Distribution Curves for Eastern United States

Figure A-2a through A-2i - Distribution Curves for Eastern United States

Distribution Curves for Hawaii (Figures A-3a through A-3c)

Methods for Developing R Distribution Curves for the Western United States

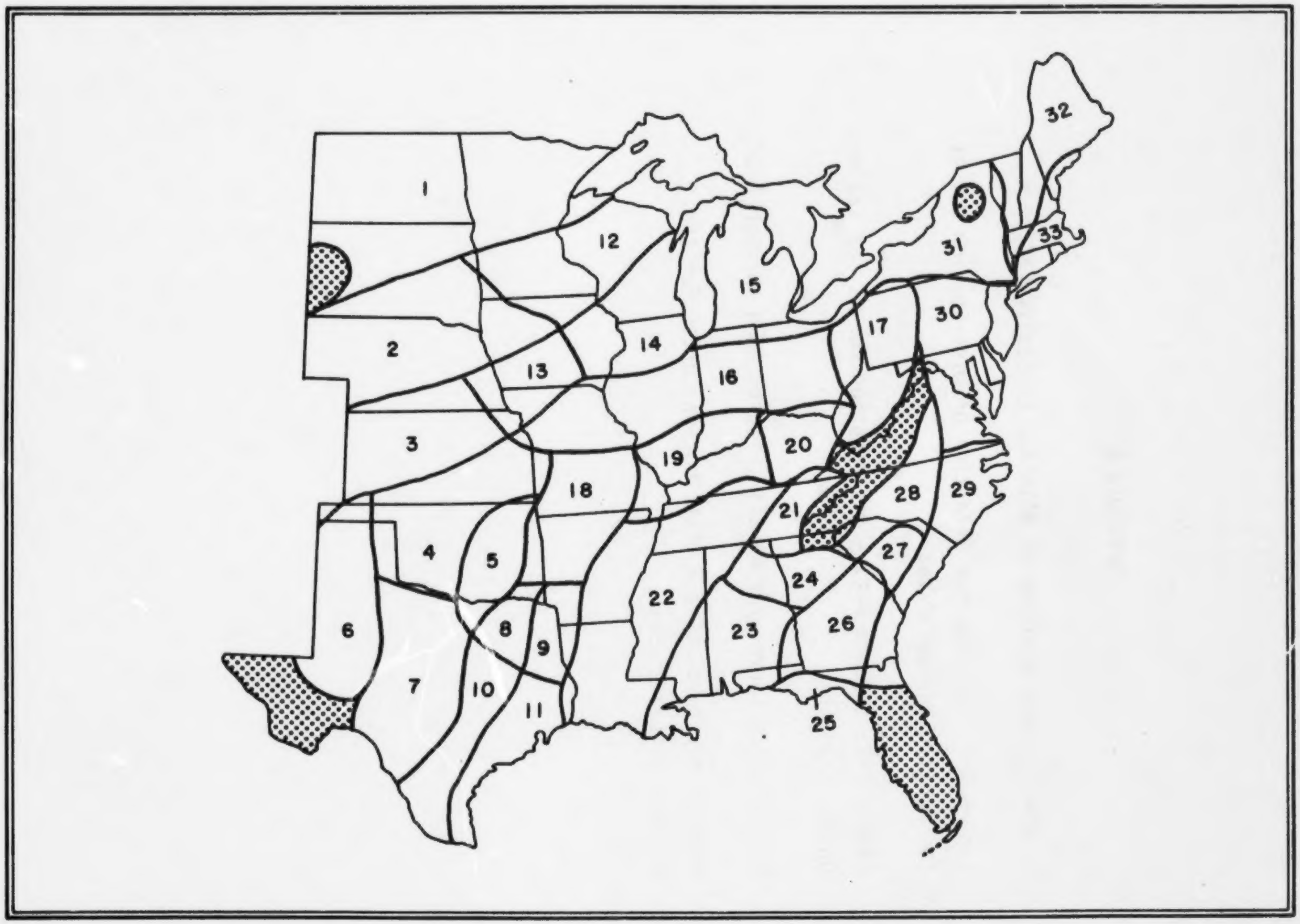
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FIGURE A-1 KEY MAP FOR SELECTION OF APPLICABLE EROSION-INDEX DISTRIBUTION CURVE (WISCHMEIR AND SMITH, 1965)

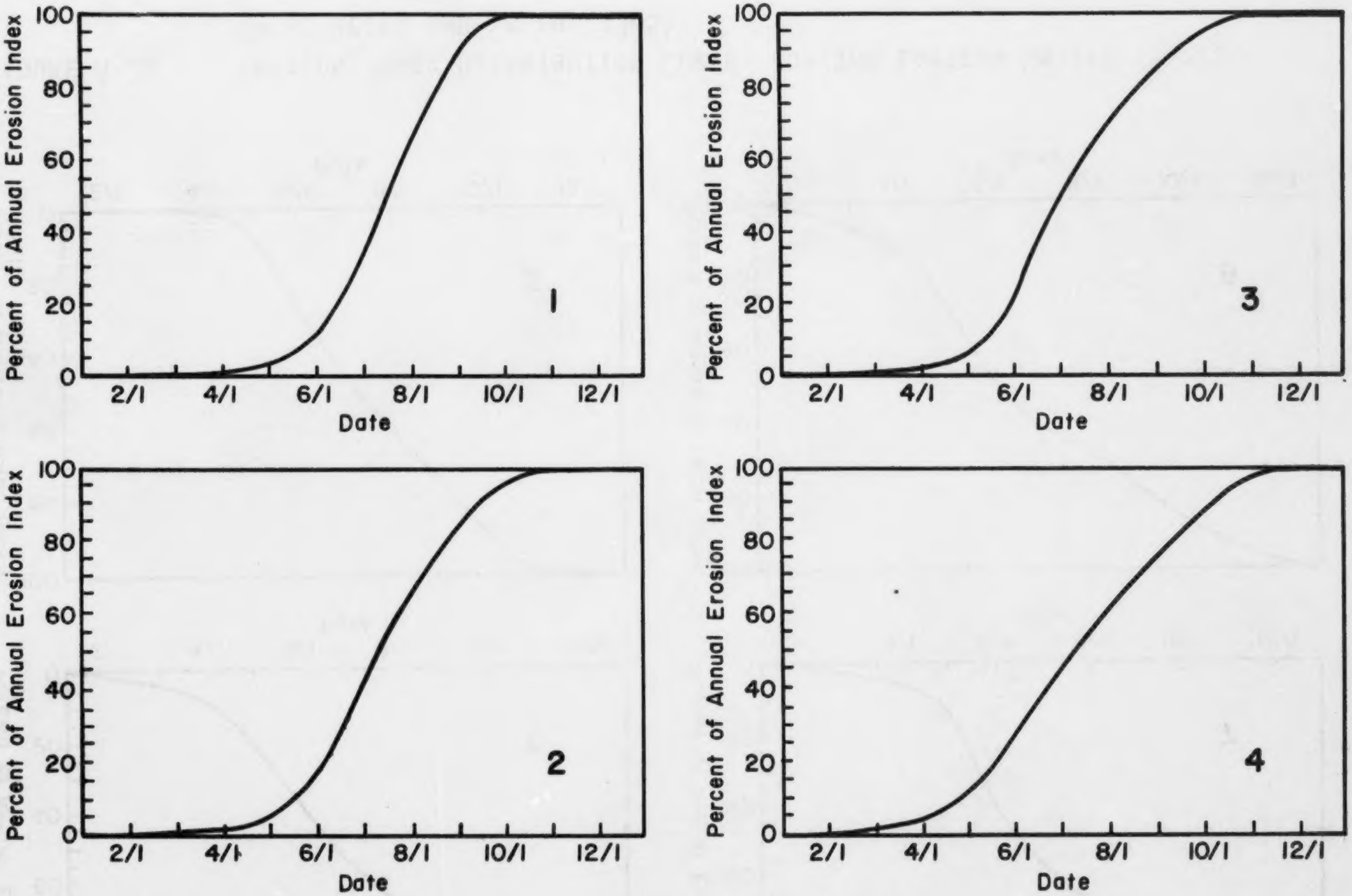

FIGURE A-2A EROSION-INDEX DISTRIBUTION CURVES FOR THE EASTERN UNITED STATES (WISCHMEIER AND SMITH, 1965)

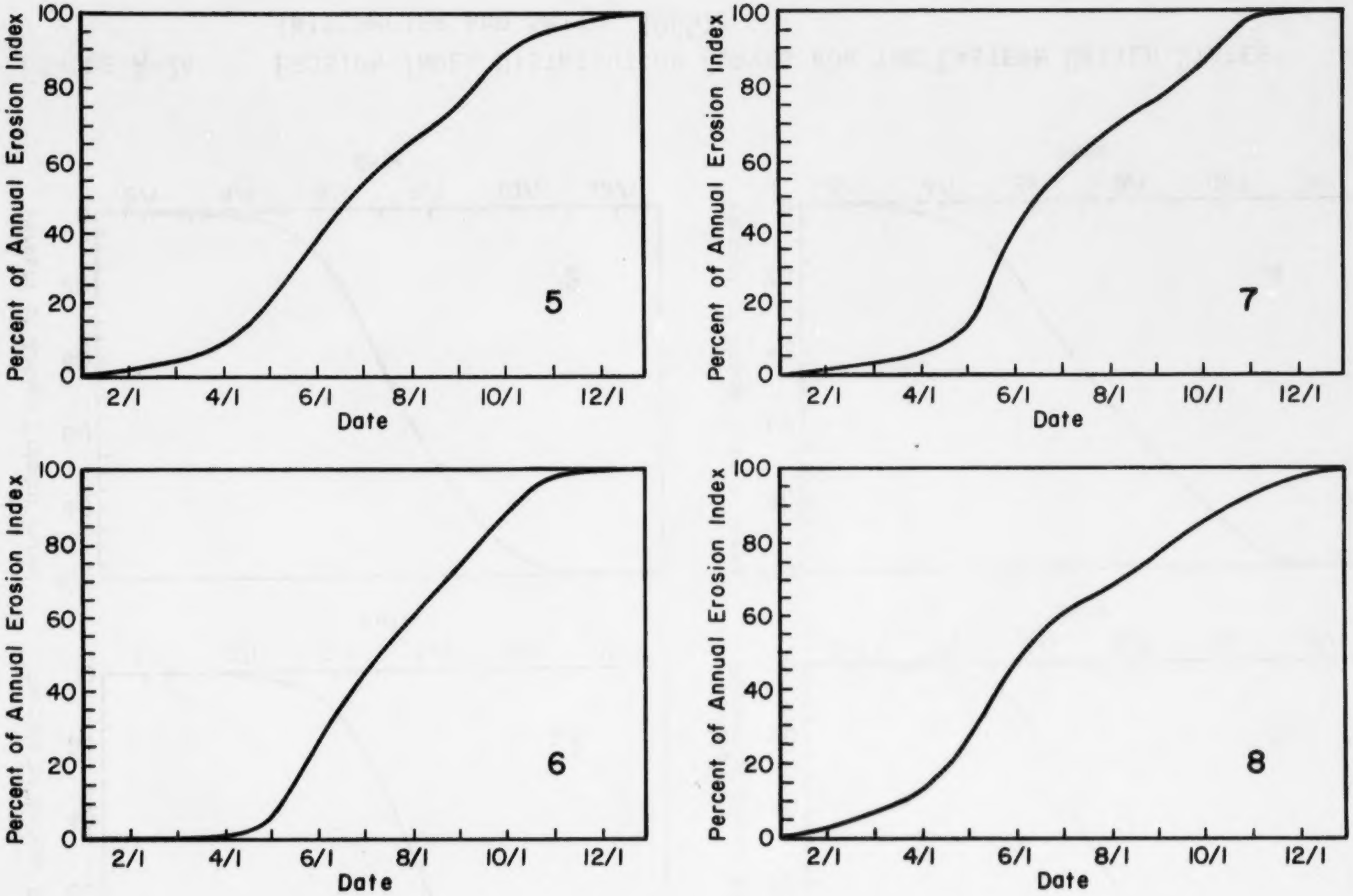

FIGURE A-2B EROSION-INDEX DISTRIBUTION CURVES FOR THE EASTERN UNITED STATES (WISCHMEIER AND SMITH, 1965)

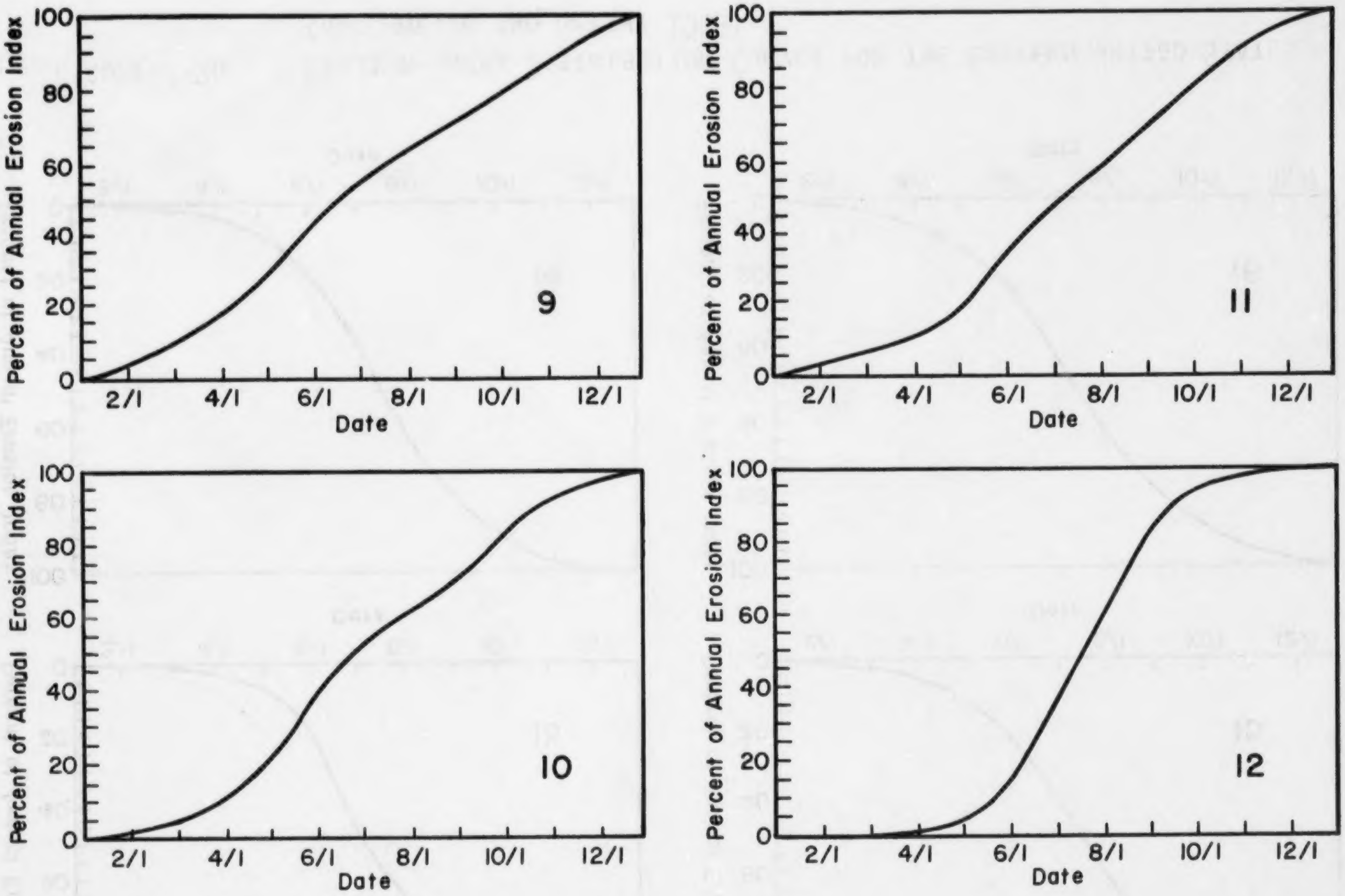

FIGURE A-2C EROSION-INDEX DISTRIBUTION CURVES FOR THE EASTERN UNITED STATES (WISCHMEIER AND SMITH, 1965)

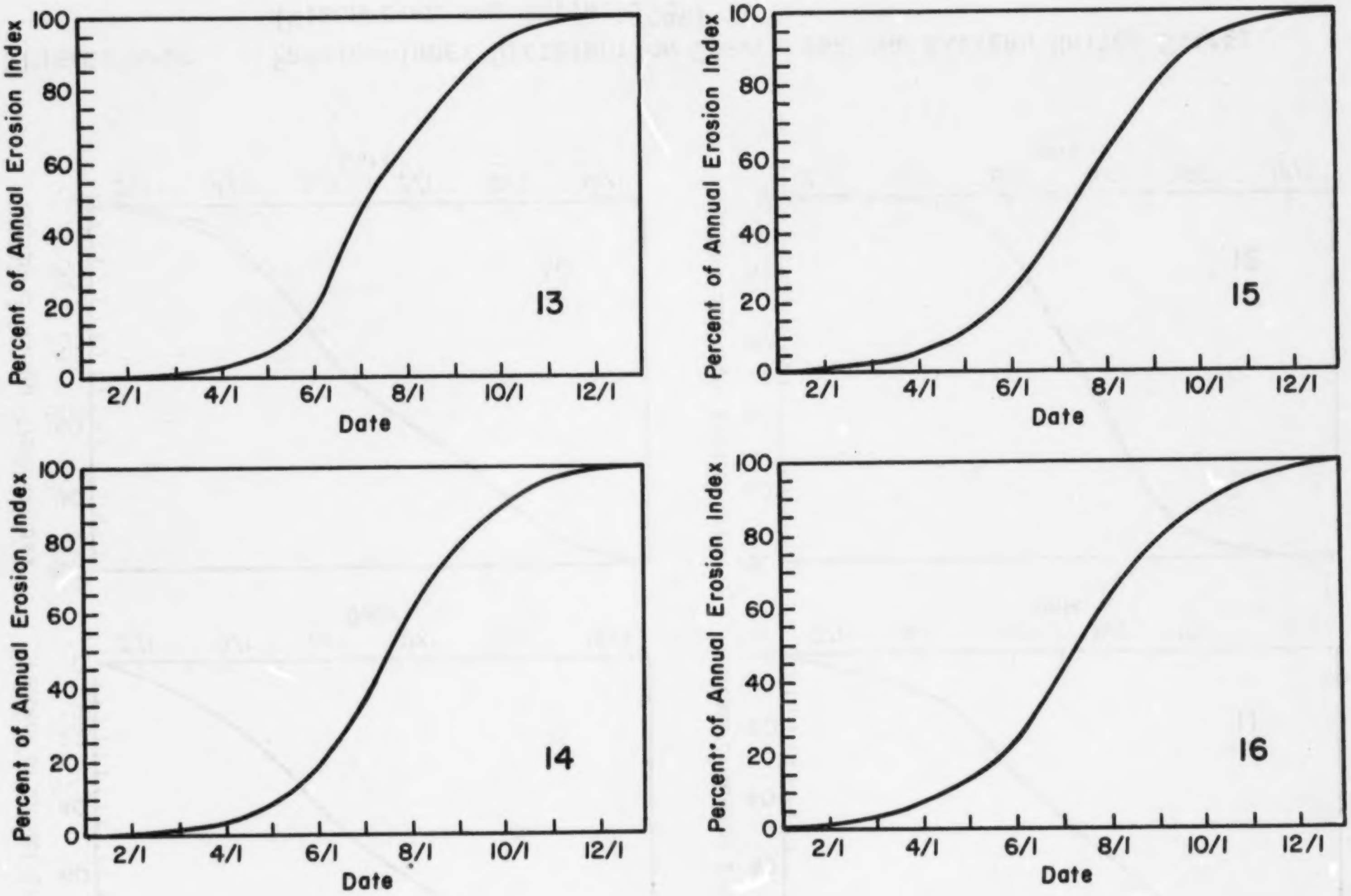

FIGURE A-2D EROSION-INDEX DISTRIBUTION CURVES FOR THE EASTERN UNITED STATES (WISCHMEIER AND SMITH, 1965)

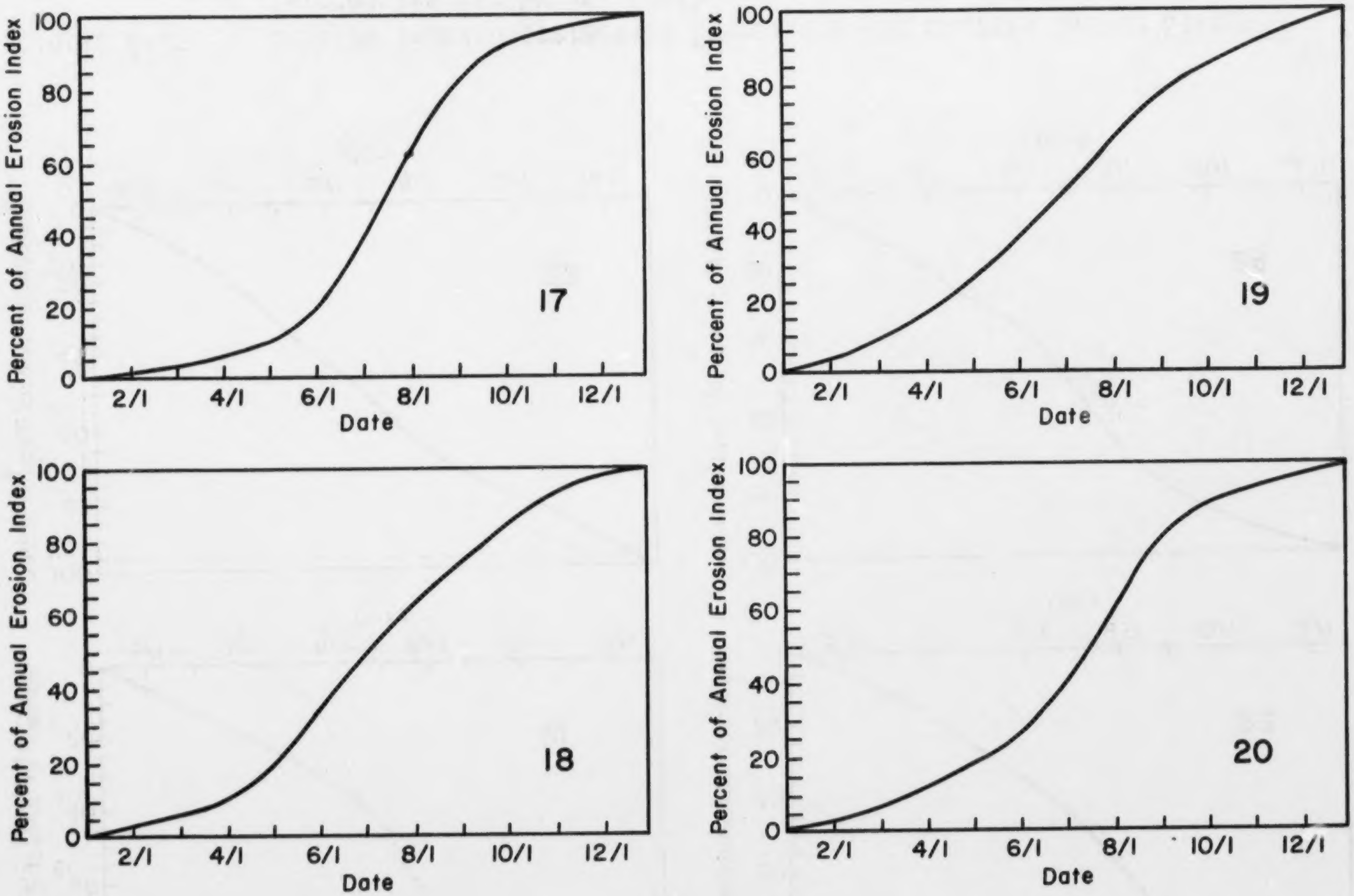

FIGURE A-2E EROSION-INDEX DISTRIBUTION CURVES FOR THE EASTERN UNITED STATES (WISCHMEIER AND SMITH, 1965)

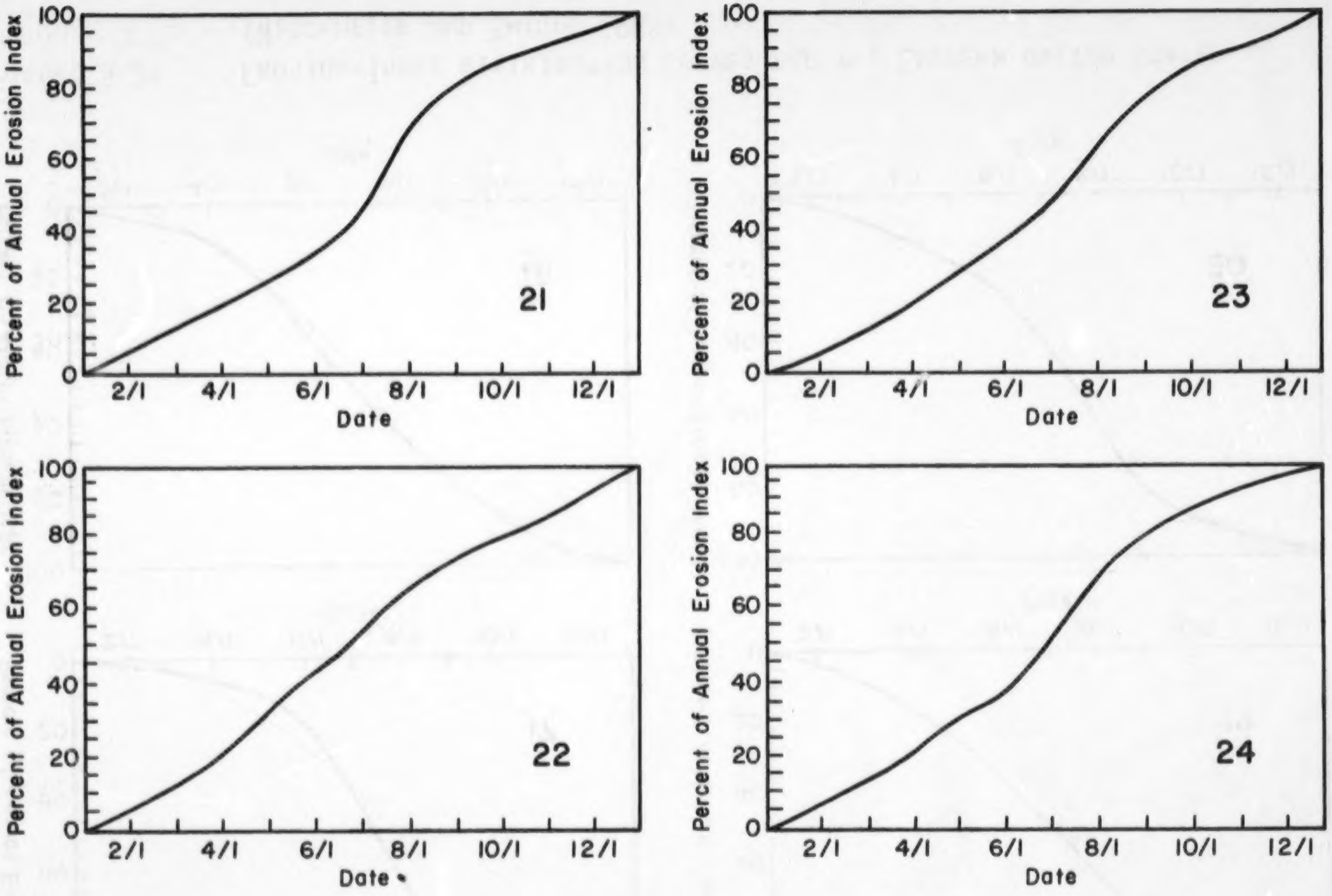

FIGURE A-2F EROSION-INDEX DISTRIBUTION CURVES FOR THE EASTERN UNITED STATES (WISCHMEIER AND SMITH, 1965)

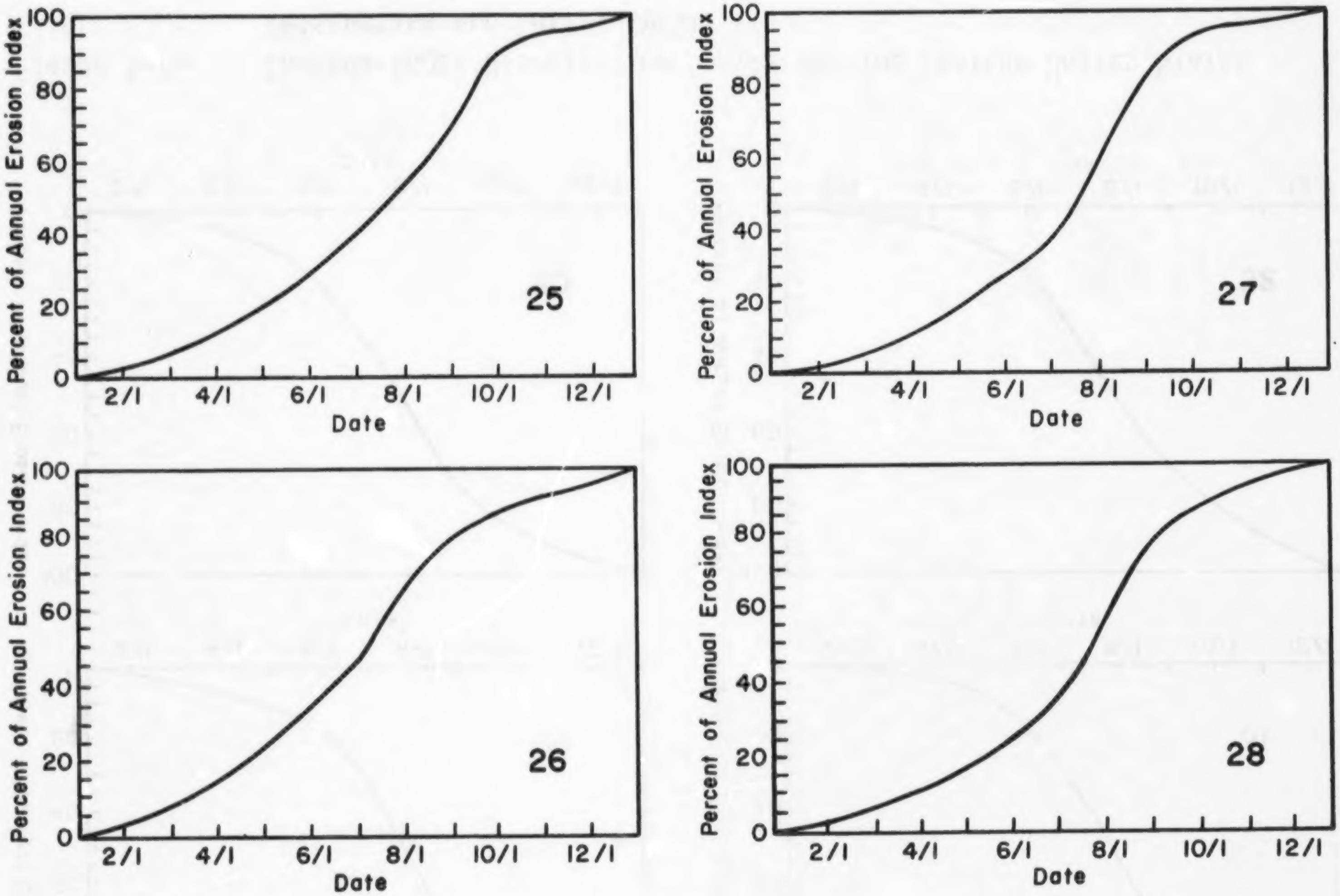

FIGURE A-2G EROSION-INDEX DISTRIBUTION CURVES FOR THE EASTERN UNITED STATES (WISCHMEIER AND SMITH, 1965)

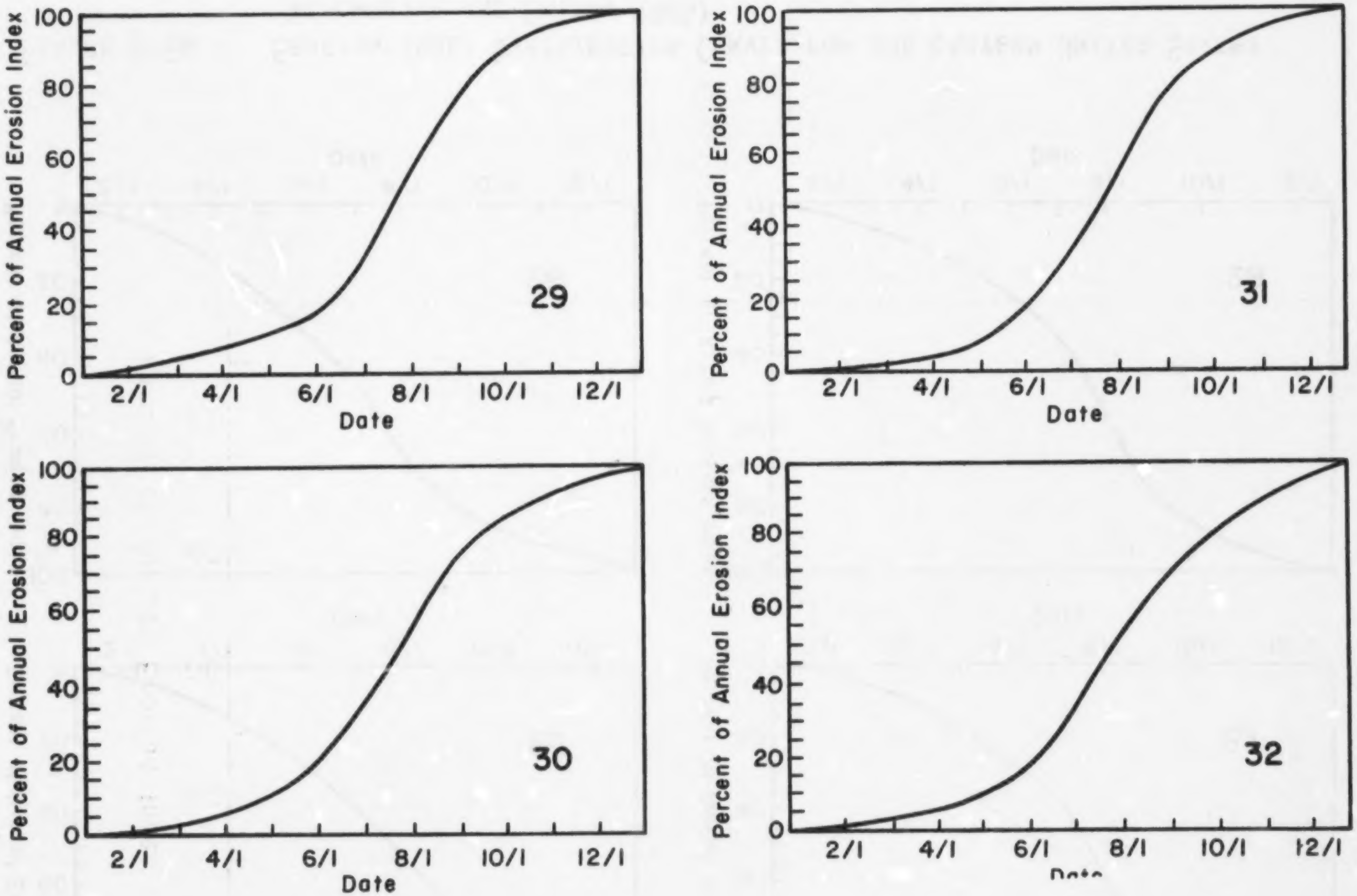

FIGURE A-2H EROSION-INDEX DISTRIBUTION CURVES FOR THE EASTERN UNITED STATES (WISCHMEIER AND SMITH, 1965)

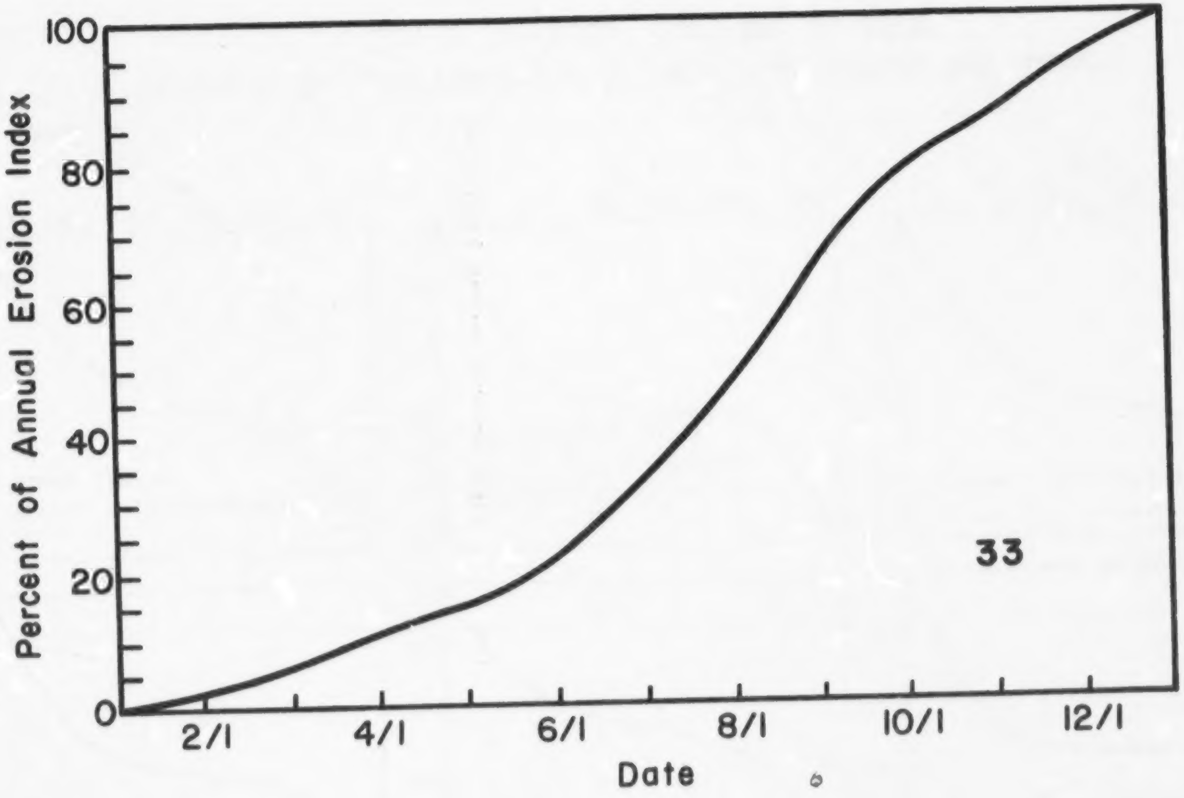

FIGURE A-21 EROSION-INDEX DISTRIBUTION CURVES FOR THE EASTERN UNITED STATES (WISCHMEIER AND SMITH, 1965)

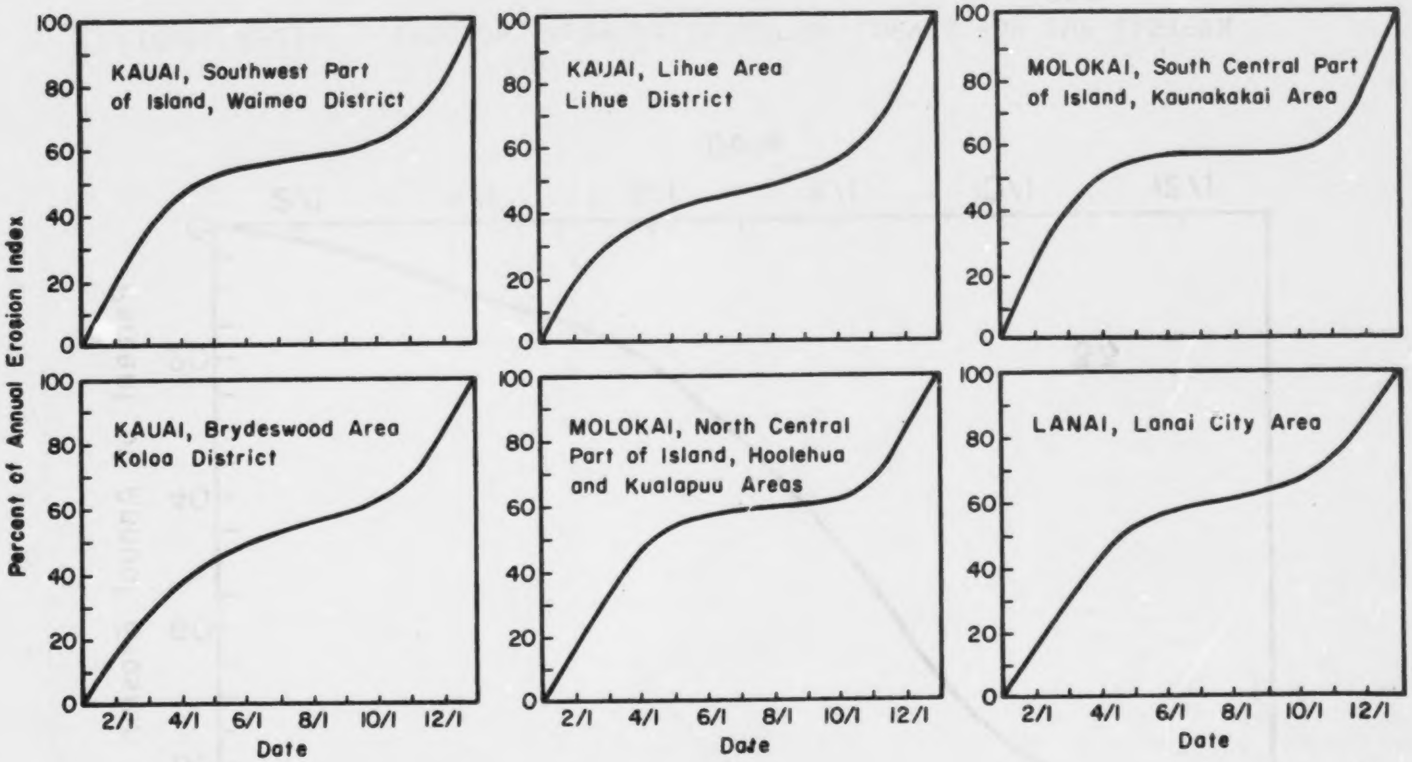

FIGURE A-3A EROSION-INDEX DISTRIBUTION CURVES FOR HAWAII
(SOILS TECHNICAL NOTE NO. 3, 1974)

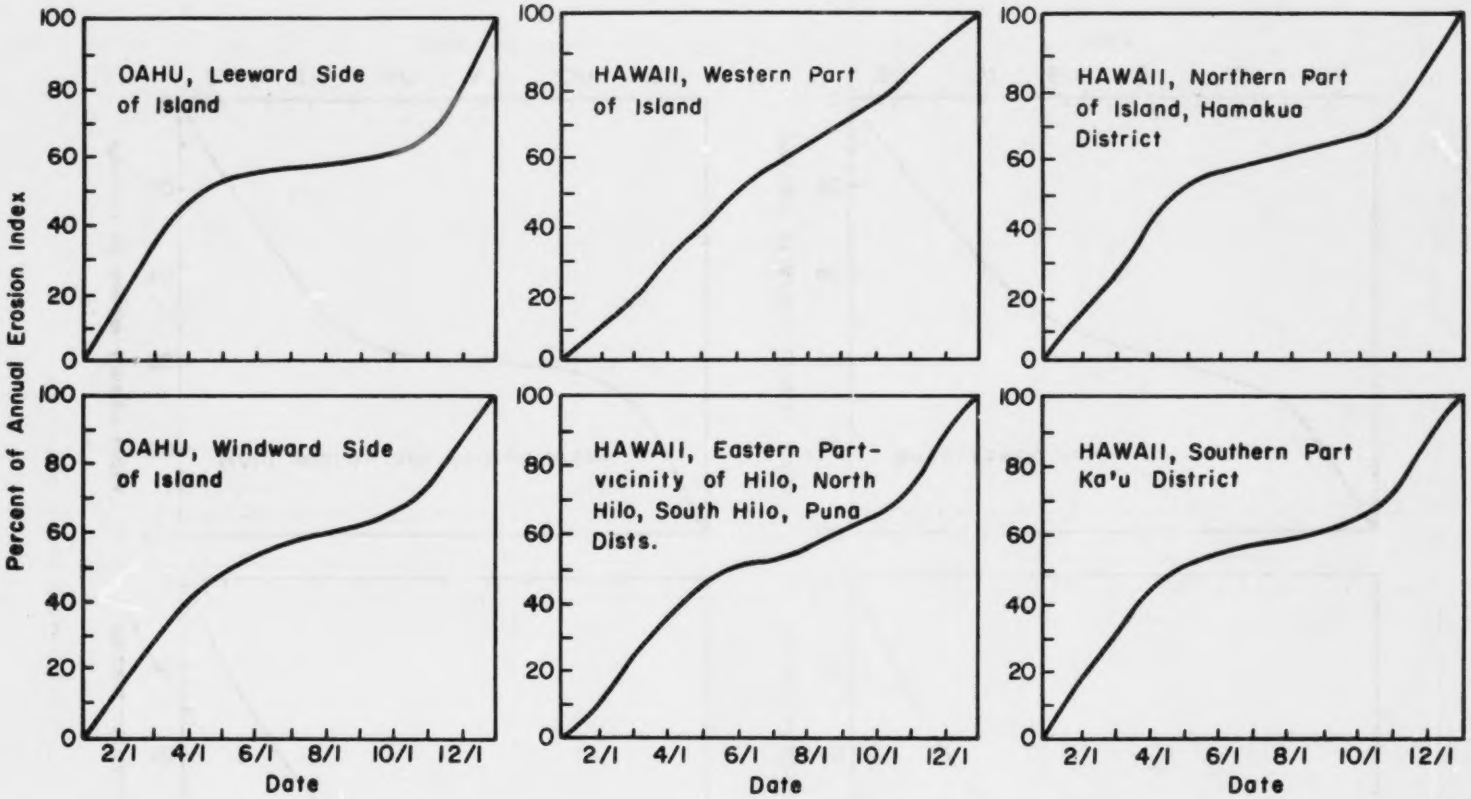

FIGURE A-3B EROSION-INDEX DISTRIBUTION CURVES FOR HAWAII (SOILS TECHNICAL NOTE NO. 3, 1974)

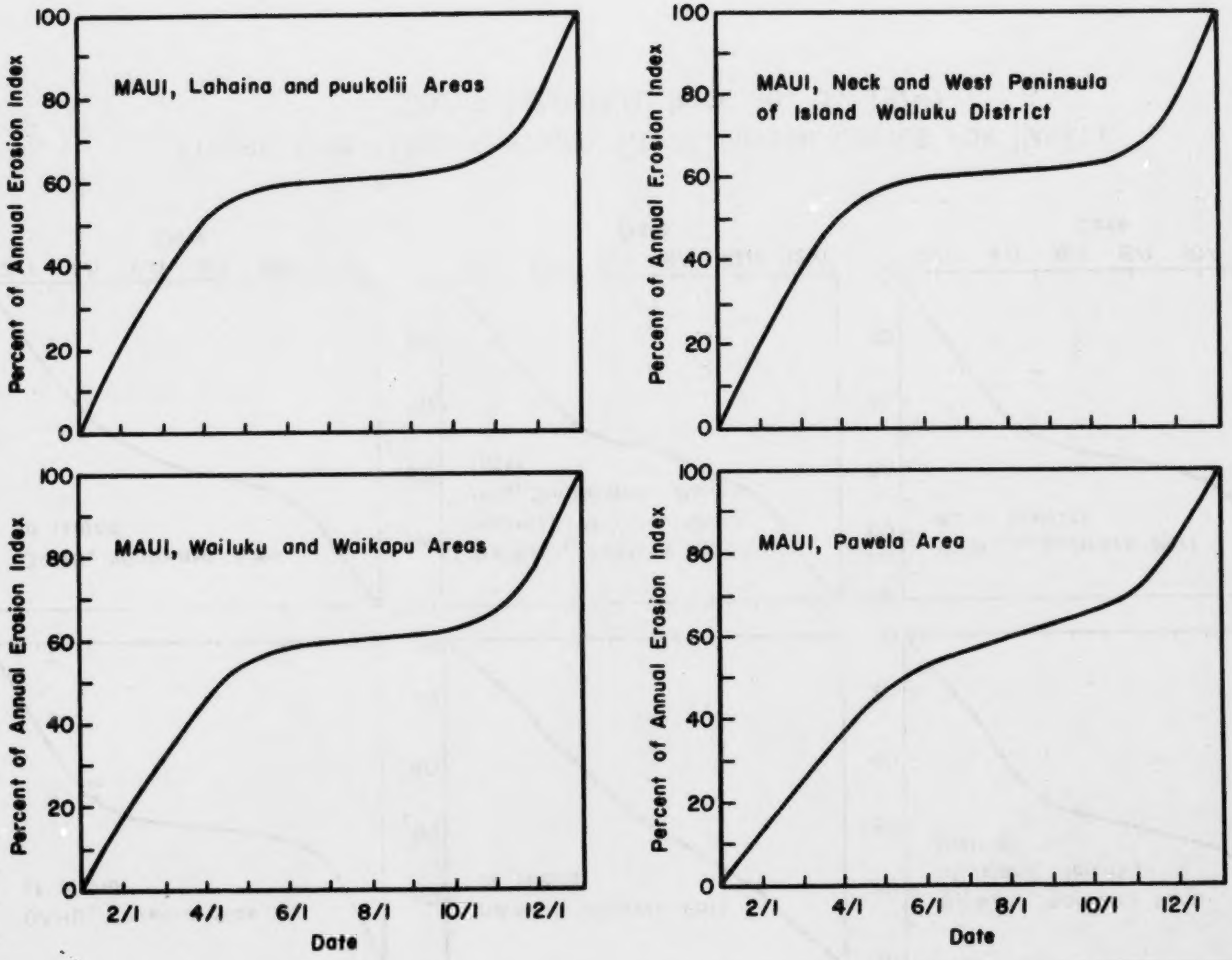

FIGURE A-3C EROSION-INDEX DISTRIBUTION CURVES FOR HAWAII
(SOILS TECHNICAL NOTE NO. 3, 1974)

METHODS FOR DEVELOPING R DISTRIBUTION CURVES FOR THE WESTERN UNITED STATES (Conservation Agronomy Technical Note No. 32, 1974)

R_s is significant in portions of this area. Divide the annual R_r for the location by the average annual precipitation to obtain a factor. Multiply each month's precipitation by this factor to obtain monthly R_r values. Add the prorated monthly R_s values to R_r for the months when snowmelt occurs, to obtain the monthly R values. Compute. the monthly accumulative percent. The following example is for Hylton, in Elko County, Nevada. The 2-6 rainfall for this area is 0.9 in. The annual R_r determined from the Type II curve on Figure III-4, is 18. Annual precipitation average is 12.72 in. Factor is 18 ÷ 12.72 = 1.42.

Monthly precipitation (water depth) for December through March is 4.92 in. $R_s = 4.92 \times 1.5 = 7.38$. This is prorated, based on local judgment to

January	10%	or	0.7
February	20%	or	1.5
March	50%	or	3.7
April	20%	or	1.5

Month (1)	Precipitation (Inches Water Depth) (2)	R_r (3)	R_s (4)	R* (5)	Cumulative R (6)	Cumulative % (7)
January	1.18	1.68	0.7	2.38	2.38	0.093
February	1.14	1.62	1.5	3.12	5.50	21.6
March	1.29	1.83	3.7	5.53	11.03	43.3
April	1.49	2.12	1.5	3.62	14.65	57.5
May	1.48	2.10	-	2.10	16.75	65.8

---

*Columns (3) + (4).

Month (1)	Precipitation (Inches Water Depth) (2)	R_r (3)	R_s (4)	R* (5)	Cumulative R (6)	Cumulative % (7)
June	0.91	1.29	-	1.29	18.04	70.9
July	0.63	0.89	-	0.89	18.93	74.4
August	0.52	0.74	-	0.74	19.67	77.3
September	0.63	0.89	-	0.89	20.56	80.8
October	1.17	1.66	-	1.66	22.22	87.3
November	0.97	1.38	-	1.38	23.60	92.7
December	1.31	1.86	-	1.86	25.46	100.

---

*Columns (3) + (4).

Values in cumulative percent column (7) are the points used in plotting the monthly R distribution curve.

For A-2, A-3, and A-4 Areas Shown in Figure III-4

R_s is not significant in most parts of these areas. Use the monthly rainfall distribution as the R distribution. Simply accumulate monthly precipitation amounts and divide each by the annual precipitation. The results obtained for each month will be the points for plotting the monthly R distribution curve.

For B-1 and C Areas Shown in Figure III-4

R_s in most parts of these areas is significant.

1. "Multipliers" are used to time average monthly precipitation amounts. Sum the results of multiplications to obtain the "factored annual precipitation". Divide the annual R_r for

the location by the "factored annual precipitation" to obtain a factor which will be used to convert monthly precipitation amounts to the monthly R values (see the previous section for A-1 area). Values of multipliers are:

Month(s)	Multipliers
January, February, March	0.1
April	1.0
May	4.0
June, July, August	7.0
September, October	2.0
November, December	0.1

2. Add the prorated R_s values to the months when the snowmelt occurs to obtain the monthly R values. Compute the monthly cumulative percents which are points used in plotting the monthly R distribution curve. The following example is for a hypothetical area which has an annual rainfall factor R_r of 25, and a R_s factor of 7.5 (4.94 x 1.5 rounded to 7.5). The 4.94 in. is total precipitation for December, January, February, and March. R_s factor is prorated to:

January	0%	or	0 in.
February	33.3%	or	2.5 in.
March	33.3%	or	2.5 in.
April	33.3%	or	2.5 in.

Month (1)	Precipitation (in.) (2)	Multiplier (3)	Factored Monthly pptn. (Col. 2 x Col. 3) (4)	Monthly R_r* (5)
January	1.33	0.1	0.13	0.11
February	1.14	0.1	0.11	0.09
March	1.35	0.1	0.13	0.11
April	1.48	1.0	1.48	1.24
May	1.43	4.0	5.72	4.80
June	1.00	7.0	7.00	5.78
July	0.80	7.0	5.60	4.69
August	0.78	7.0	5.46	4.58
September	0.85	2.0	1.70	1.43
October	1.14	2.0	2.28	1.91
November	0.92	0.1	0.09	0.08
December	1.12	0.1	0.11	0.09
Total	13.34		29.81	25.0

Month (1)	Monthly R_s (6)	Monthly R $=R_r + R_s$ (7)	Cumulative R (8)	Cumulative % (9)
January	-	0.11	0.1	-
February	2.5	2.59	2.7	8
March	2.5	2.66	5.4	17
April	2.5	3.74	9.1	28
May	-	4.80	13.9	43
June	-	5.87	19.8	61
July	-	4.69	24.5	75
August	-	4.58	29.0	89
September	-	1.43	30.5	94
October	-	1.91	32.4	99
November	-	0.08	32.4	100
December	-	0.09	32.5	100
Total	7.5	32.5		

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*In this example, the calculated factor value is 0.84 (25 + 29.81). Monthly R_r is obtained by multiplying each "factored monthly pptn." with 0.84.

For B-2 Area Shown in Figure III-3

In this area, no R_s values are needed. Follow the same procedure and use the same set of multipliers as the preceding section for areas B-1 and C, except that steps for obtaining monthly R_s values are not used. The cumulative R and cumulative percent are computed from monthly R_r (column 5 in the preceding example).

REFERENCES FOR APPENDIX A

Conservation Agronomy Technical Note No. 32, U.S. Department of Agriculture, Soil Conservation Service, West Technical Service Center, Portland, Oregon, September, 1974.

Soils Technical Note No. 3, U.S. Department of Agriculture, Soil Conservation Service, Honolulu, Hawaii, May, 1974.

Wischmeier, W.H., and Smith, D.D., "Predicting Rainfall--Erosion Losses from Cropland East of the Rocky Mountains," Agricultural Handbook 282, U.S. Department of Agriculture, Agriculture Research Service, May, 1965.

APPENDIX B

METHODS FOR PREDICTING SOIL ERODIBILITY INDEX K

Nomograph for Predicting K Values of Surface Soils Using Chemical and Physical Parameters.

Nomograph for Predicting K Values of High Clay Subsoils Using Chemical Mineralogical and Physical Parameters.

NOMOGRAPH FOR PREDICTING K VALUES OF SURFACE SOIL

In 1971 Wischmeier, et al. (1971) presented a soil erodibility nomograph derived from statistical analysis of 55 soil types. Five soil parameters are included in the nomograph to predict erodibility: percent silt plus very fine sand; percent sand greater than 0.10 millimeter; organic matter content; soil structure; and permeability. Values of the parameters may be obtained from routine laboratory determinations and standard soil profile descriptions.

The nomograph is reproduced here as Figure B-1.

Description of Factors (Water Resources Administration, 1973)

Grain Size Distribution

Grain size distribution has a major influence on a soil's erodibility: the greater the silt content, the greater the soil's erodibility; the smaller the sand content, the greater the soil's erodibility.

Particles in the very fine sand classification behave more like silt than sand. Therefore, the percentage of very fine sand should be subtracted from the total percentage of sand and added to the percentage of silt.

Organic Matter

The percentage of organic matter was determined, in work by Wischmeier, et al., by the Walkley-Black method (Walkley and Black, 1934). The organic matter content is approximately 1.72 times the percent carbon. Soil erodibility decreases as organic matter content increases.

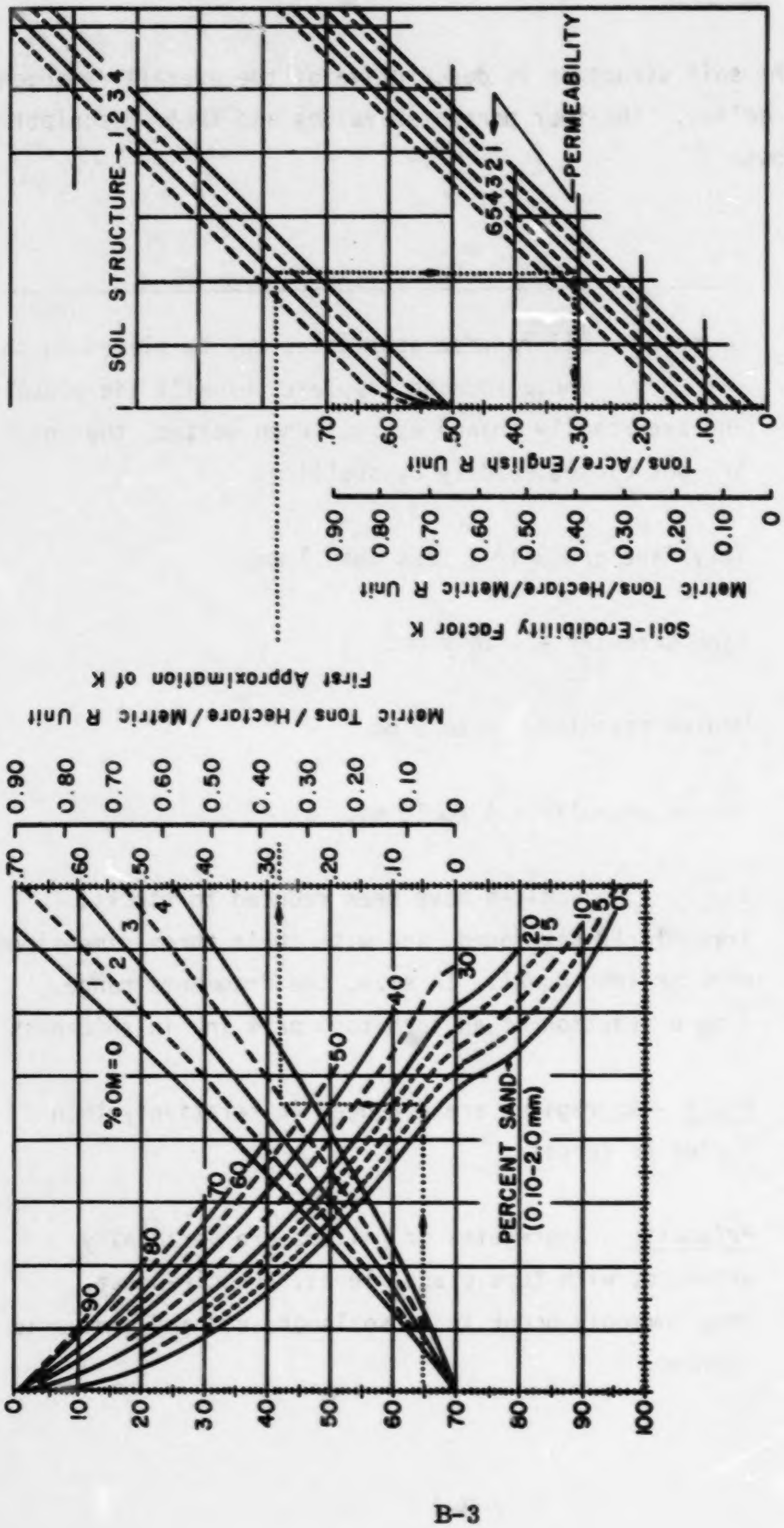


FIGURE B-1 SOIL ERODIBILITY NOMOGRAPH (WISCHMEIER ET AL., 1971)

Soil Structure

The soil structure is descriptive of the overall arrangement of the soil solids. The four parameter values and their descriptions are as follows:

Parameter Value	Descriptions
	Granular - All rounded aggregates may be placed in this category. These rounded complexes usually lie loosely and are readily shaken apart. When wetted, the voids are not closed readily by swelling.
1	Very fine granular - less than 1 mm.
2	Fine granular - 1 to 2 mm.
3	Medium granular - 2 to 5 mm.
3	Coarse granular - 5 to 10 mm.
4	Blocky - Aggregates have been reduced to blocks, irregularly six-faced, and with their three dimensions more or less equal. In size, the fragments range from a fraction of an inch to 3 or 4 in. in thickness.
4	Platy - Aggregates are arranged in relatively thin plates or lenses.
4	Prismatic - Aggregates or pillars are vertically oriented, with tops plane, level, and clean cut. They commonly occur in subsoils of arid and semi-arid regions.

Parameter Value	Descriptions
4	Columnar - Aggregates or pillars are vertically oriented, with rounded tops. They commonly occur when the soil profile is changing and the horizons are degrading.
4	Massive - Soil units are very large, irregular, featureless as far as characteristic aggregates are concerned.

Soil Permeability

Soil permeability is the ability of the soil to transmit water. Since different soil horizons vary in permeability, the relative permeability classes refer to the soil profile as a whole. The relative permeability classes are as follows:

Class	Permeability Rates in In/Hour	
1	Rapid	over 6.0
2	Moderately rapid	2.0 to 6.0
3	Moderate	0.6 to 2.0
4	Moderately slow	0.2 to 0.6
5	Slow	0.06 to 0.2
6	Very slow	less than 0.06

Reading the Nomograph

Entry values for all of the nomograph curves, except permeability class, are for the upper 6 or 7 in. of soil. For soils in cuts, the entry values are for the upper 6 or 7 in. of the newly exposed layer. In reading the nomograph, interpolate linearly between adjacent curves when the entry data do not coincide with the plotted curves of percent sand or percent organic matter. The percent of coarse fragments may be significant and is not included in the nomograph. Therefore, reduce the value of K read from the nomograph by 10% for soils with stratified subsoils that include layers of small stones or gravel without a seriously impeding layer above them.

Enter the left scale of the nomograph with the appropriate percent silt plus very fine sand, move horizontally to intersect the correct percent-sand curve (interpolating to the nearest percent), vertically to the correct organic matter curve, and then horizontally to the right scale for first approximation of soil erodibility.

For soils having a fine granular structure and moderate permeability, the value of K can be obtained directly from this scale. However, if the soil is other than of fine granular structure, or permeability is other than moderate, it is necessary to proceed to the second part of the nomograph, horizontally to intersect the correct structure curve, vertically downward to the permeability curve, and horizontally to the soil erodibility index scale.

NOMOGRAPH FOR PREDICTING K VALUES OF HIGH CLAY SUBSOILS

Subsoils are commonly heavier in texture than the surface soils. In addition, subsoils likely have aggregating agents that are very much different from those found in surface soils and the degree of aggregation is known to have a profound influence on erodibility.

From an EPA study (Roth, et al., 1974) conducted at Purdue University, a multiple linear regression equation and nomograph were developed which can be used to estimate the erodibility factor, K, of many high clay soils. Multiple regression analysis revealed that amorphous iron, aluminum and silicon hydrous oxides serve as soil stabilizers in subsoils (whereas, organic matter is the major stabilizer in surface soils). The nomograph was developed from the multiple linear regression equation relating the erodibility factor to the soil texture factor, M, the amount of CDB (citrate-dithionite-bicarbonate) extractable iron and aluminum oxides, and the amount of CDB extractable silica.

The equation used to derive the nomograph was:

$$K_{pred} = 0.32114 + 2.0167 \times 10^{-4} M - 0.14440 (\% Fe_2O_3 + \% Al_2O_3) - 0.83686 (\% SiO_2)$$

where

K_{pred} = predicted K value of subsoil

M = soil texture factor, defined by percent new silt (percent new silt + percent new sand). "New" silt has 2 to 100 μm mean diameter. "New" sand has 100 to 2,000 μm mean diameter.

$\% Fe_2O_3$ = percent CBD extractable iron oxide of soil.

$\% Al_2O_3$ = percent CDB extractable aluminum oxide of soil.

$\% SiO_2$ = percent CDB extractable silica in soil.

The nomograph for estimating the erodibility factor, K, of high clay subsoils is reproduced in Figure B-2.

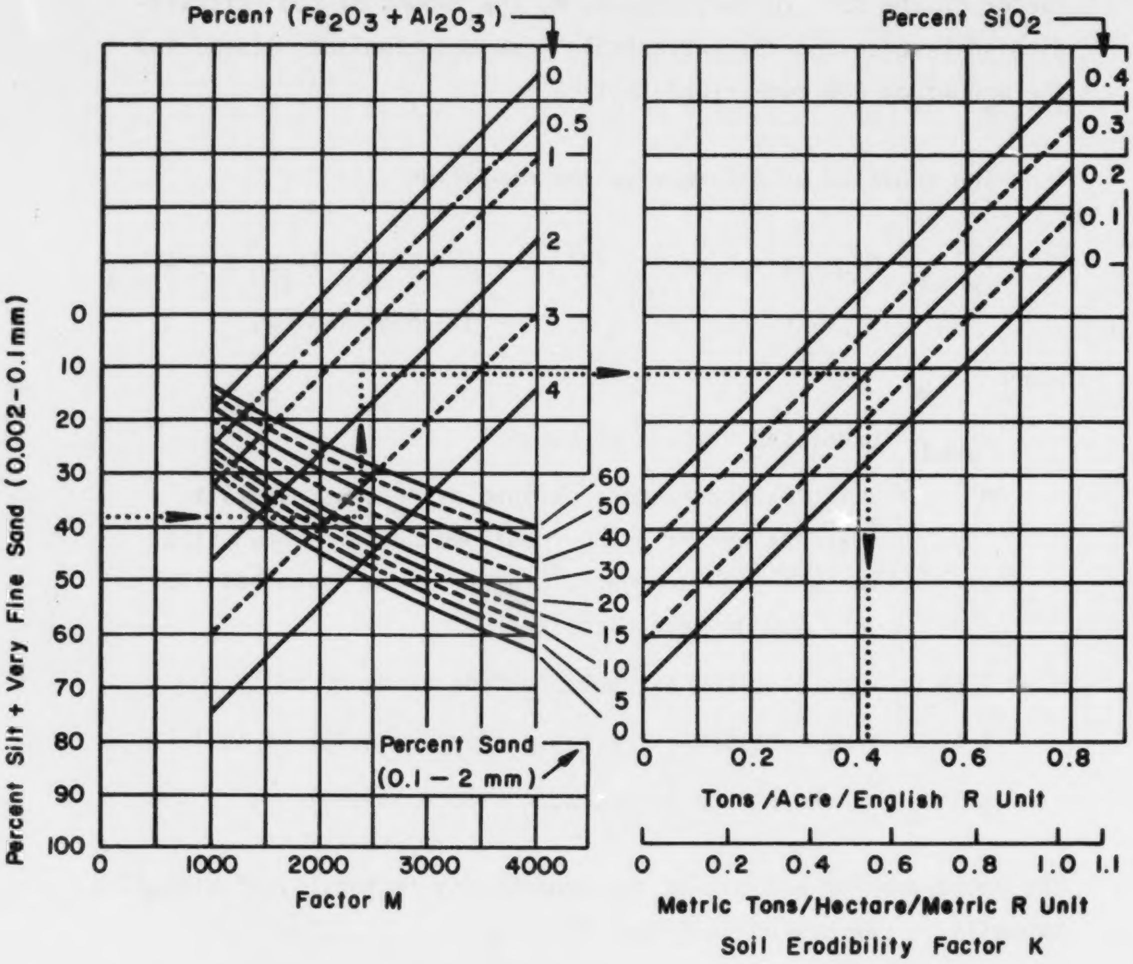


FIGURE B-2 NOMOGRAPH FOR ESTIMATING THE ERODIBILITY FACTOR K OF HIGH CLAY SUBSOILS (ROTH ET AL., 1974)

REFERENCES FOR APPENDIX B

Roth, C.B., Nelson, D.W., and Romkens, M.J.M., 1974. "Prediction of Subsoil Erodibility Using Chemical, Mineralogical, and Physical Parameters," for the U. S. Environmental Protection Agency (EPA-660/2-74-043), Washington, D.C.

Walkley, A., and Black, I.A., 1934. "An Examination of the Degtjareff Method for determining Soil Organic Matter," Soil Sci., 37, pp. 29-38.

Wischmeier, W.H., Johnson, C.B., and Cross, B.U., 1971. "A Soil Erodibility Nomograph for Farmland and Construction Sites," J. Soil and Water Conservation, 26:189-193. "Technical Guide to Erosion and Sediment Control Design (Draft)," 1973. Water Resources Administration, Maryland Department of Natural Resources, Annapolis, Maryland.

APPENDIX C

STREAM AND RIVER DATA

Table C-1 Stream and River Reaeration and Deoxygenation Rates (From Hydroscience, 1971)

Figure C-1 Deoxygenation Parameter $(k_L H^2/\nu)$ versus the Reynolds Number and Froude Number $(\rho UH/\mu)/(U\sqrt{gH})$ (From Bansal, 1975)

Table C-2 Deoxygenation Rate Constants (From Bansal, 1975)

Table C-3 Alluvial Channel and Sediment Data (From Schumm, 1960)

TABLE C-1

STREAM AND RIVER REAERATION AND DEOXYGENATION RATES (FROM HYDROSCIENCE, 1971)

River Name	Category	Depth (ft.)	Area (ft^2)	Flow (cfs)	Velocity (fps)	k_a @20^{o}C (1/day)	k_L @20^{o}C (1/day)
Grand River (Michigan)	Shallow	1.9	320.0	295.	0.92	4.5	0.59
Clinton River (Michigan)	Shallow	1.58	44.6	33.	0.72	5.9	3.37
Truckee River (Nevada)	Shallow	1.67	150.	180.	1.20	5.6	0.36
		1.67	150.	195.	1.30	5.7	0.36
		1.67	150.	271.	1.81	6.6	0.96
Flint River (Michigan)	Shallow	2.1	210.	134.	0.64	3.5	0.56
		2.6	200.	174.	0.83	3.9	0.63
		2.6	400.	174.	0.44	3.1	0.69
		1.7	290.	204.	0.73	5.0	0.69
		1.9	400.	204.	0.51	2.2	0.69
Jackson River (Virginia)	Shallow	3	365.	100.	0.27	4.1	1.25
N. Branch Potomac River (Md., W.Va.)	Shallow	2	100.	100.	1.0	9.0	0.40

TABLE C-1 (continued)

River Name	Category	Depth (ft.)	Area (ft^2)	Flow (cfs)	Velocity (fps)	k_a @20°C (1/day)	k_L @20°C (1/day)
Clarion River	Shallow	1		1 - 10.			3
(Penna.)		1.9			0.55	2.26	
South River		1-2		35.			2
Ivel River	Shallow	1.21		4.86	0.14	2.35	
(England)		1.51		4.15	0.14	2.06	
		1.09		3.87	0.13	3.20	
		1.50		15.40	0.37	2.37	
		1.08		4.86	0.16	4.57	
		0.38		4.15	0.15	2.09	
		1.12		3.87	0.13	1.18	
		1.46		15.40	0.38	3.18	
		1.31		15.40	0.47	6.18	
		2.44		10.07	0.23	0.90	
		2.03		10.07	0.22	1.66	
Lark River	Shallow	1.74		10.94	0.28	0.78	
(England)		1.47		10.94	0.37	2.12	
		1.82		36.20	0.50	1.41	
		2.41		36.20	0.43	0.31	

TABLE C-1 (continued)

River Name	Category	Depth (ft.)	Area (ft^2)	Flow (cfs)	Velocity (fps)	k_a @20°C (1/day)	k_L @20°C (1/day)
Derwent River (England)		0.72		21.60	1.37	31.80	
		0.89		21.60	1.19	24.53	
		0.85		21.60	1.07	34.57	
Black Beck River (England)	Shallow	0.40		2.70	0.44	25.59	
		0.40		2.70	0.56	28.34	
		0.39		2.70	0.63	22.80	
		0.60		17.70	1.83	49.17	
		0.69		17.70	1.81	30.77	
		1.00		17.70	1.54	18.46	
St. Sunday's Beck (Eng.)	Shallow	0.82		19.10	1.07	21.05	
		0.78		19.10	1.27	16.06	
Yewdale Beck (England)	Shallow	0.64		5.10	0.46	12.04	
		0.48		5.10	0.60	30.32	
		0.72		17.30	1.16	18.90	
		0.66		17.30	1.31	20.25	
		0.67		17.30	1.30	17.09	
		0.69		17.30	1.25	19.16	
Elk River (Penna.)	Shallow	0.9			0.97	5.84	

TABLE C-1 (continued)

River Name	Category	Depth (ft.)	Area (ft^2)	Flow (cfs)	Velocity (fps)	k_a @20^oC (1/day)	k_L @20^oC (1/day)
Mohawk River	Shallow	3	143.			.07-4.0	.23
Mohawk River (New York)	Medium	15	3,800.	800.	.21		.40
North Branch Susquehanna	Medium	4	1,700.	1,000.	0.60	1.5	0.35
New River (Virginia)	Medium	5	1,720.	1,200.	0.70	1.04	2.5 0.5
Wabash (Indiana)	Medium	5-7		1,000-5,000			.40
Clinch River (T.V.)	Medium	3.27		3,300.	3.07	2.27	
		5.09		4,500	3.69	1.44	
		4.42		3,190	3.10	.98	
		6.14		5,890	2.68	.50	
		5.65		5,910	2.78	.74	
		7.17		5,930	2.64	1.13	
Holston (T.V.)	Medium	11.41		10,385	2.92	.28	
		2.12		3,230	2.47	3.36	
		2.93		6,400	3.44	2.79	
		4.54		14,085	4.65	1.57	
		9.50		10,440	3.94	.46	
		6.29		6,540	2.51	.39	

TABLE C-1 (continued)

River Name	Category	Depth (ft.)	Area (ft^2)	Flow (cfs)	Velocity (fps)	k_a @20°C (1/day)	k_L @20°C (1/day)
Holston River (T.V.)	Medium	7.52		10,500	3.15	.27	
		7.07		10,500	3.30	.55	
		5.44		5,590	3.11	.54	
		8.06		11,930	4.28	.60	
		3.98		952	2.73	1.25	
Fr. Broad (T.V.)	Medium	9.38		12,010	2.41	.27	
		10.19		17,120	3.06	.23	
		3.29		44,105	2.40	1.88	
		4.74		8,775	3.46	.84	
		5.72		12,455	4.02	.88	
		6.98		17,270	52	.91	
		4.29		4,150	85	1.00	
		6.01		8,775	.75	.55	
		7.16		12,455	.23	.98	
		9.49		17,270	3.71	.25	
Wautaga River (T.V.)	Medium	3.42		3,112	5.0	43.	5.6
Hiwassee River (T.V.)	Medium	3.02		1,145	3.05	11.	1.7
		2.83		1,145	3.91	21.	3.2
Ohio River	Deep	32	43,000	6,000	.14		.06

TABLE C-1 (continued)

River Name	Category	Depth (ft.)	Area (ft^2)	Flow (cfs)	Velocity (fps)	k_a @20°C (1/day)	k_L @20°C (1/day)
Upper Hudson (Troy, N.Y.-Saugerties)	Deep	17.5	6,000	3,000	0.5	.34	.125
		21.0	6,750	4,500	1.5	.34	.165
Lower Sacramento River	Deep	15-20	8,000	10,000	1.5	.28	.40
						.15	
						.24	
Upper James River (Va.)	Deep	15.5	11,500	1,800	0.16	.15	.48
			14,500	2,600	0.18	.14	.30
			14,500	9,000	0.63	.12	.31
			14,000	7,500	0.53	.13	.41
			14,500	4,500	0.31	.22	.39
			13,500	3,800	0.28	.24	.38
			15,000	1,350	0.13	.22	.43
Illinois River	Deep	10-12		8,000			.07
		9.2			1.37	.225	
		9.0			1.57	.269	
		8.9			1.63	.224	

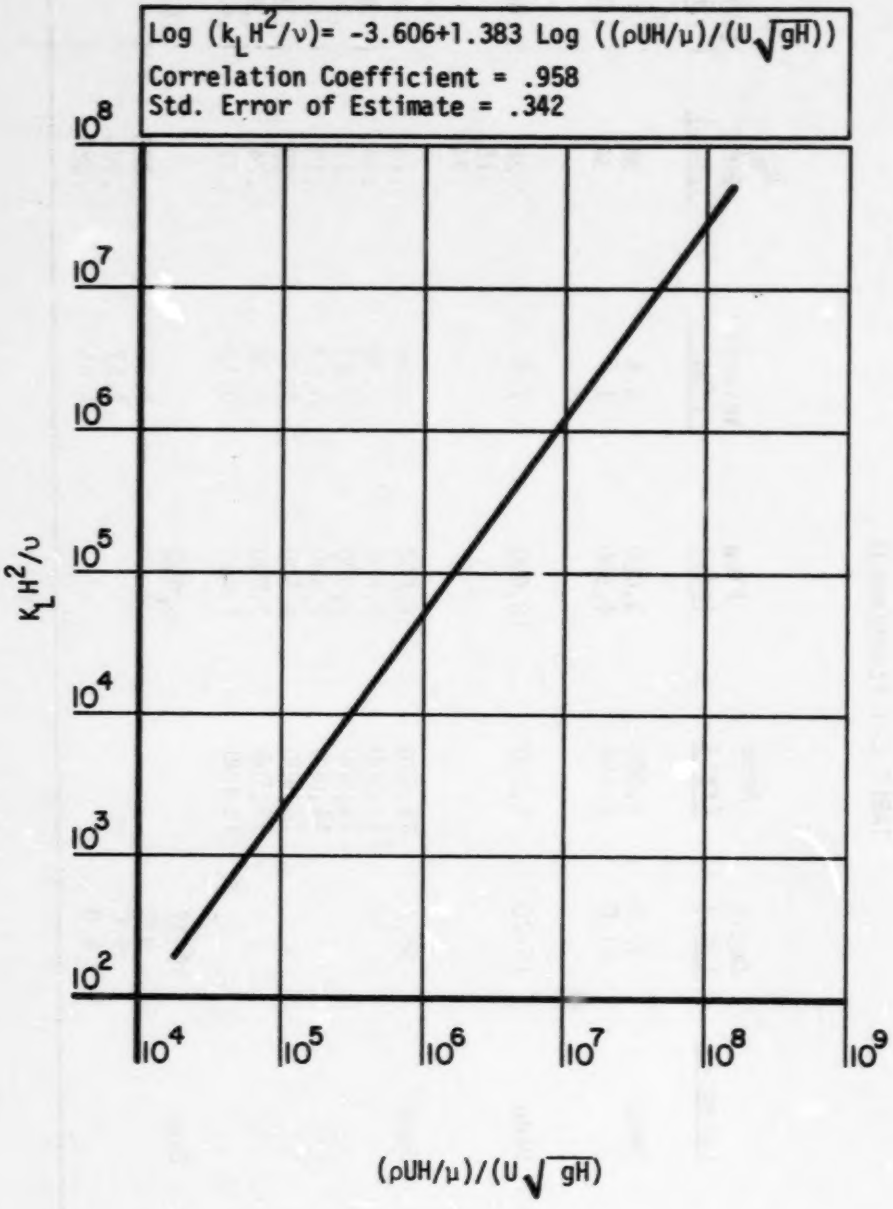


FIGURE C-1 DEOXYGENATION PARAMETER ($K_L H^2/\nu$) VERSUS THE REYNOLDS NUMBER AND FROUDE NUMBER ($\rho UH/\mu$)/$U\sqrt{gH}$) (FROM BANSAL, 1975)

TABLE C-2

DEOXYGENATION RATE CONSTANTS (FROM BANSAL, 1975)

				k_L rates base e/day	
Discharge cfs	X-section area sq ft.	Top Width ft.	Temp °C	Observed	Estimated (From Fig. C-1)
Kansas River at Bonner Springs, Kansas					
15,200	4,300	770	25	.02	.258
2,160	1,200	505	28	.12	.249
2,090	1,170	500	25	.12	.242
2,440	1,300	525	24	.24	.241
1,300	850	450	9	.02	.204
828	550	415	5	.16	.190
632	425	405	9	.26	.195
1,080	710	432	14	.17	.213
Kansas River at Lecompton, Kansas					
1,750	750	725	27	.19	.232
1,360	590	660	32	.15	.239
2,060	880	774	28	.35	.236
2,300	1,000	757	10	.30	.201
1,040	450	592	6	.47	.184
793	350	538	0	.23	.169
1,170	500	620	16	.05	.206
Kansas River at Topeka, Kansas					
3,040	1,450	468	22	.08	.241
1,460	700	437	31	.07	.248
1,800	865	447	27	.1	.243
2,690	1,285	466	18	.37	.230
1,900	910	450	7	.14	.200
764	365	405	7	.06	.189
631	310	368	11	.23	.196
608	290	364	15	.10	.204

TABLE C-2 (continued)

				k_L rates base e/day	
Discharge cfs	X-section area sq ft.	Top Width ft.	Temp °C	Observed	Estimated (From Fig. C-1)
Republic River below Milford, Kansas					
258	184	196	24	.18	.224
657	412	251	28	.19	.242
609	392	249	24	.07	.283
201	140	166	14	.25	.203
15	10	68	1	.14	.153
36	26	77	0	.23	.161
249	177	192	16	.29	.208
Smoky Hill River at Enterprise, Kansas					
215	122	119	27	.09	.232
100	57	69	32	.26	.238
373	211	131	24	.16	.234
146	83	91	6	.32	.193
113	65	75	0	.14	.173
210	120	118	15	.17	.207
157	88	97	14	.24	.204
Smoky Hill River at New Cambridge, Kansas					
185	87	86	27	.09	.232
67	35	79	31	.19	.225
734	252	88	29	.27	.255
Solomon River at Niles, Kansas					
61	65	53	28	.21	.237
35	40	49	30	.19	.234
117	125	61	21	.06	.231

TABLE C-2 (continued)

				k_L rates base e/day	
Discharge cfs	X-section area sq ft.	Top Width ft.	Temp. ^oC	Observed	Estimated (From Fig. C-1)
Kansas River at Wamego, Kansas					
890	390	413	27	.23	.231
1,540	670	468	26	.13	.236
2,530	1,080	540	27	.30	.244
1,470	730	462	15	.26	.214
680	300	395	0	.28	.171
535	225	381	2	.11	.172
483	190	375	7	.06	.181
Big Blue River at Tuttle Creek, Kansas					
1,060	1,050	194	22	.21	.251
162	70	76	24	.14	.224
90	42	62	27	.23	.225
810	490	191	23	.28	.240
961	1,000	194	8	.6	.217
232	108	92	1	.37	.179
50	30	52	6	.15	.181
50	30	52	13	.2	.195
Kansas River at Manhattan (Fort Riley), Kansas					
1,250	4,250	533	26	.26	.268
559	1,750	493	32	.15	.265
1,200	4,050	530	26	.10	.267
511	367	247	25	.09	.234

TABLE C-2 (continued)

Discharge cfs	X-section area sq ft.	Top Width ft.	Temp. ^{o}C	k_L rates base e/day Observed	Estimated (From Fig. C-1)
Solomon River at Glen Elder, Kansas					
44	38	49	28	.20	.229
47	40	49	28	.23	.23
79	58	53	24	.10	.227
30	30	33	7	.35	.21
48	41	49	0	.34	.172
33	31	36	0	.37	.173
30	30	33	5	.35	.185
Saline River at Tescott, Kansas					
8.3	14	19.4	2.7	.37	.226
5.8	10	17.8	28	.25	.224
132	75	30	21	.26	.235
Smoky Hill River at Mentor, Kansas					
138	88	83	24	.10	.226
35	20	81.5	26	.15	.207
675	288	92.5	26	.42	.25
Smoky Hill River at Langley, Kansas					
77	60	57	23	.42	.224
147	85	67	27	.51	.236
493	210	90	24	.14	.240
249	122	75	23	.14	.232
14	15	20	7	.20	.186
21	21	23	1	.28	.176
18	19	22	6	.29	.186
81	60	58	11	.33	.199

TABLE C-2 (continued)

					k_L rates base e/day	
	Discharge cfs	X-section area sq ft.	Top Width ft.	Temp. ^oC	Observed	Estimated (From Fig. C-1)
Grand River, Michigan						
	295	320	168.4	20	.59	.228
Clinton River, Michigan						
	33	44.6	28.22	20	3.37	.225
Truckee River, Nevada						
	180	150	89.8	20	.36	.226
	195	150	89.8	20	.36	.226
	271	150	89.8	20	.96	.226
Flint River, Michigan						
	134	210	100		.56	.230
	174	200	76.9	20	.63	.233
	174	400	153.8	20	.69	.233
	204	296	170.6	20	.69	.226
	204	400	210.5	20	.69	.228
Jackson River, Virginia						
	100	365	122	20	1.25	.236
North Branch Potomac River (Maryland, West Virginia)						
	100	100	50	20	.4	.229
North Branch Susquehanna						
	1,000	1,700	425	20	.35	.241

TABLE C-2 (continued)

Discharge cfs	X-section area sq ft.	Top Width ft.	Temp. ^{o}C	k_L rates base e/day Observed	k_L rates base e/day Estimated (From Fig. C-1)
New River, Virginia					
1,200	1,720	344	20	.5	.245
Upper Hudson, Troy, New York					
3,000	6,000	343	20	.125	.269
4,500	6,750	321	20	.165	.273
Lower Sacramento River					
10,000	8,000	457	20	.4	.269
Upper James River, Virginia					
1,800	8,000	742	20	.48	.267
2,600	11,500	935.5	20	.30	.267
9,000	14,500	935.5	20	.31	.267
7,500	14,000	903.2	20	.41	.267
4,500	14,500	935.2	20	.39	.267
3,800	13,500	871	20	.38	.267
1,350	15,000	967.7	20	.43	.267
Cooper River, South Carolina					
10,000	40,000	1,000	20	.3	.286
Savannah River, Georgia and South Carolina					
7,000	10,000	1,000	20	.3	.258
6,800	40,000	1,428.6	20	.3	.279

TABLE C-2 (continued)

					k_L rates base e/day	
	Discharge cfs	X-section area sq. ft.	Top Width ft.	Temp. °C	Observed	Estimated (From Fig. C-1)
South New Jersey						
	23	2,500	208.3	20	.2	.262
Compton Creek, New Jersey						
	10	1,000	69	20	.23	.265
	10	790	75	20	.23	.259

TABLE C-3

ALLUVIAL CHANNEL AND SEDIMENT DATA (FROM SCHUMM, 1960)

Location	Median grain size, D_{50} (mm)	Silt-clay in bank (percent)	Silt-clay in channel (percent)	Weighted mean silt-clay M (percent)	Width (feet)	Depth (feet)	Width depth ratio (F)	Gradient (S)	Mean annual flood (cfs)	Mean annual discharge (cfs)	Drainage area (sq mi)
Smoky Hill-Kansas Rivers system											
Willow Creek near Cheyenne Wells, Colo	1.10	72	3	16	15	1.7	8.8	------	------	------	------
Smoky Hill River near Arapahoe, Colo	.85	49	3	6.1	65	2.3	28	0.003	------	30(est)	------
Smoky Hill River near Sharon Springs, Kans	.41	25	4	4.5	200	2.5	80	------	------	------	------
Smoky Hill River at Russell Springs Kans	1.30	21	2	2.4	263	3.0	88	------	------	------	------
Smoky Hill River at Gove, Kans	.80	63	3	4.3	226	2.5	90	------	------	------	------
Smoky Hill River near Arnold, Kans	.93	30	2	2.4	345	2.5	138	------	5,800	65.2	5,220
Smoky Hill River near Russell, Kans	.81	76	1	5.3	115	3.5	33	.00066	8,000	215	6,965
Smoky Hill River at Dorrence, Kans	1.30	69	.5	4.4	130	4.0	33	.0007	------	------	------
Smoky Hill River near Kanopolis (Longley), Kans	.63	96	4	14	92	5.5	17	.0005	9,200	314	7,857
Smoky Hill River near Bridgeport (Lindsborg), Kans	.40	85	3	13	69	5.0	14	------	6,750	340	8,110
Smoky Hill River at Abilene, Kans	.023	97	87	89	125	18	7	------	11,500	1,254	18,830
Smoky Hill River near Junction City, Kans	1.20	90	.5	6	153	5.0	31	.0004	13,000	1,454	19,900
Kansas River at Wamego, Kans	.70	93	1	3.8	636	10	64	.0008	39,000	4,398	55,240
Kansas River near Topeka, Kans	.75	57	.5	3	800	18	44	.0005	48,000	5,155	56,710
Republican River System											
Arikaree River near Arikaree, Colo	1.10	82	3	4.7	206	2.2	94	------	------	------	------
Arikaree River at Haigler, Nebr	.25	65	3	8	68	3.0	23	0.002	3,500	19.6	1,460
Republican River near Stratton, Nebr	.38	31	3	3.4	400	3.0	133	------	------	------	------
South Fork Republican River near Benkleman, Nebr.	.48	44	1.5	3.4	100	2.3	43	.002	4,500	56.8	2,580
Republican River near Benkleman, Nebr	.25	23	6	6.7	123	2.5	49	.003	2,175	105	4,770
Republican River near McCook, Nebr	.52	88	.5	4.4	115	2.7	43	.009	------	------	5,760
Republican River near Bostwick(Hardy), Nebr	.63	29	1	2.8	154	5.8	31	.0008	12,000	843	22,400
Republican River at Concordia, Kans	.70	34	.2	1.4	250	5.0	50	.0007	13,000	------	23,540
Republican River at Junction City, Kans	.60	59	1	3.4	300	6.5	46	.0007	15,000	1,000	24,900
Powder River System											
South Fork Powder River near Kaycee, Wyo	0.63	71	9	11.3	119	2.3	52	0.004	3,900	35	1,150
Middle Fork Powder River above Kaycee, Wyo	22.0	60	14	20	35	2.5	14	.005	574	58	450
Middle Fork Powder River near Kaycee, Wyo	.40	69	15	23	47	4.4	11	.0015	1,630	133	980
Powder River below Arvada, Wyo	.21	70	4	6.5	175	3.5	50	.0011	------	------	------
Powder River near Locate, Mont	.42	58	13	15	234	4.5	52	------	9,400	639	------
Crazy Woman Creek near Arvada, Wyo	.50	75	2	17	33	4.4	7.5	------	1,150	40	956
Little Powder River at Broadus, Mont	4.10	82	5.5	22	40	5.5	7.3	------	1,280	39	------
Bighorn River near Kane, Wyo	.16	35	20	21	220	8.5	26	------	16,100	2,888	15,900
Badwater Creek near Lysite, Wyo	.43	47	3	6.7	50	2.3	22	.0037	------	------	------
Badwater Creek at Lysite, Wyo	.24	58	5	7.3	109	2.5	44	.0037	------	------	------
Owl Creek near Thermonolis, Wyo	.21	69	2	14	35	3.9	9.0	.0015	585	34	484
Cottonwood Creek at Winchester, Wyo	1.0	15	8	8.4	133	3.5	38	------	------	------	------
Gooseberry Creek at Pulliam, Wyo	8.0	45	2.5	5.7	59	2.4	25	.006	311	10	371
Graybull River near Basin, Wyo	.50	72	7	9.9	134	3.1	43	.0015	3,140	178	1,130
Bates Creek near Alcova, Wyo	.90	63	14	18	69	2.8	25	.0035	500	16	377

TABLE C-3 (continued)

Location	Median grain size, D_{50} (mm)	Silt-clay in bank (percent)	Silt-clay in channel (percent)	Weighted mean silt-clay M (percent)	Width (feet)	Depth (feet)	Width depth ratio (F)	Gradient (S)	Mean annual flood (cfs)	Mean annual discharge (cfs)	Drainage area (sq mi)
Sage Creek, S.Dak.:											
(2)	0.06	93	55	73	16	7	2.3	0.0055	-----	-----	1.7
(3)	.06	93	68	79	20	7	2.9	.0045	-----	-----	3.4
(4)	.12	96	40	54	31	5	6.2	.0045	-----	-----	9.5
Sand Creek, Nebr:											
(4)	.72	70	14	23	75	7	10.7	.0015	-----	-----	17.9
(5)	.73	60	15	22	65	7	9.3	.003	-----	-----	22.2
(6)	.35	65	10	20	36	4	9.0	.001	-----	-----	22.5
Arroyo Calabasas, N. Mex.:											
(A)	.84	18	3	4.1	79	3	26.3	.013	-----	-----	3,8
(6)	.50	26	3	4.8	92	4	23.0	.009	-----	-----	24.2
(7)	.75	16	5	5.8	100	4	25.0	.011	-----	-----	25.8
Bayou Gulch, Colo.:											
(3)	.58	13	4	4.4	130	3	43	.010	-----	-----	19.7
(6)	.55	6	4	4.1	128	1.5	85	.016	-----	-----	22.9
Medano Creek, Colo.:											
(1)	.24	.5	1	1	340	2	170	.017	-----	-----	25.8
(2)	.24	.5	1	1	800	3	267	.019	-----	-----	26.1
(3)	.24	.5	.5	.5	820	2.5	328	.016	-----	-----	28.8
Saline River at Russell, Kans.	3.57	93	5.7	11	93	3	31	-----	4,300	88.3	1,502
Paradise Creek near Paradise, Kans.	.50	74	8	30	32	7.8	4.1	.001	1,300	11.1	212
North Fork Solomon River near Downs Kans.	.80	89	1.2	16	82	8.6	9.6	.0006	8,000	151	2,390
Solomon River at Bennington(Niles),Kans.	.41	90	4	11	112	5	22	-----	7,000	558	6,770
Prairie Dog Creek at Norton, Kans.	.90	82	1.5	19	45	6.2	7.2	.0005	2,600	33.2	721
Sappa Creek at Stamford, Nebr.	.60	97	2	23	43	6.0	7.2	.0013	1,800	111	3,840
Sappa Creek at Beaver City, Nebr.	.70	96	17	43	26	6.3	4.1	.003	1,350	39.1	1,500
Beaver Creek at Beaver City,Nebr.	.70	95	2	19	40	4.5	8.9	.001	1,000	28.8	2,060
Beaver Creek at Ludell, Kans.	1.10	95	2.5	35	28	8.0	3.5	.001	450	12.5	1,460
Frenchman Creek at Hamlet, Nebr.	.27	93	8.7	31	36	6.5	5.5	.0013	850	101	1,480
Blackwood Creek at Culbertson, Nebr.	.02	91	75	81	27	8.4	3.2	.0021	690	5.8	290
Red Willow Creek near Red Willow,Nebr.	.11	91	30	45	45	7.1	6.3	.001	2,220	43.1	400
South Loup River near Cumro, Nebr.	.25	80	9.4	16	143	7.3	19.6	.003	2,080	165	1,340
Niobrara River near Colclesser,Nebr.	.33	47	2	3.3	224	3.4	65.9	.003	-----	-----	2,000
White River at Chadron,Nebr.	.15	86	32.5	56	25	10.0	2.5	-----	880	20.4	676
White River at Interior,S.Dak.	.50	89	2	5.3	293	5.8	50.6	.002	10,900	302	------
Cheyenne River at Edgemont,S.Dak.	.75	56	.6	3	221	5.0	44.2	.0025	3,660	113	7,143

APPENDIX D

IMPOUNDMENT THERMAL PROFILES

This appendix contains thermal profile plots for a variety of impoundment sizes and geographic locations throughout the U.S. The locations are arranged in alphabetical order. Within each location set, the plots are ordered by depth and hydraulic residence time. An index to the plots is provided on pages D-2 and D-3, and the modeling approach is described in Appendix F.

INDEX TO APPENDIX D

Page

ATLANTA, GEORGIA
20' Initial Maximum Depth D-4
40' Initial Maximum Depth D-14
75' Initial Maximum Depth D-24
100' Initial Maximum Depth D-34
200' Initial Maximum Depth D-44

BILLINGS, MONTANA
20' Initial Maximum Depth D-54
40' Initial Maximum Depth D-64
75' Initial Maximum Depth D-74
100' Initial Maximum Depth D-84
200' Initial Maximum Depth D-94

BURLINGTON, VERMONT
20' Initial Maximum Depth D-104
40' Initial Maximum Depth D-114
75' Initial Maximum Depth D-124
100' Initial Maximum Depth D-134
200' Initial Maximum Depth D-144

FLAGSTAFF, ARIZONA
20' Initial Maximum Depth D-154
40' Initial Maximum Depth D-164
75' Initial Maximum Depth D-174
100' Initial Maximum Depth D-184
200' Initial Maximum Depth D-194

FRESNO, CALIFORNIA
20' Initial Maximum Depth D-204
40' Initial Maximum Depth D-214
75' Initial Maximum Depth D-224
100' Initial Maximum Depth D-234
200' Initial Maximum Depth D-244

MINNEAPOLIS, MINNESOTA
20' Initial Maximum Depth D-254
40' Initial Maximum Depth D-264
75' Initial Maximum Depth D-274
100' Initial Maximum Depth D-284
200' Initial Maximum Depth D-294

INDEX TO APPENDIX D (Continued)

Page

SALT LAKE CITY, UTAH

20' Initial Maximum Depth D-304
40' Initial Maximum Depth D-314
75' Initial Maximum Depth D-324
100' Initial Maximum Depth D-334
200' Initial Maximum Depth D-344

SAN ANTONIO, TEXAS

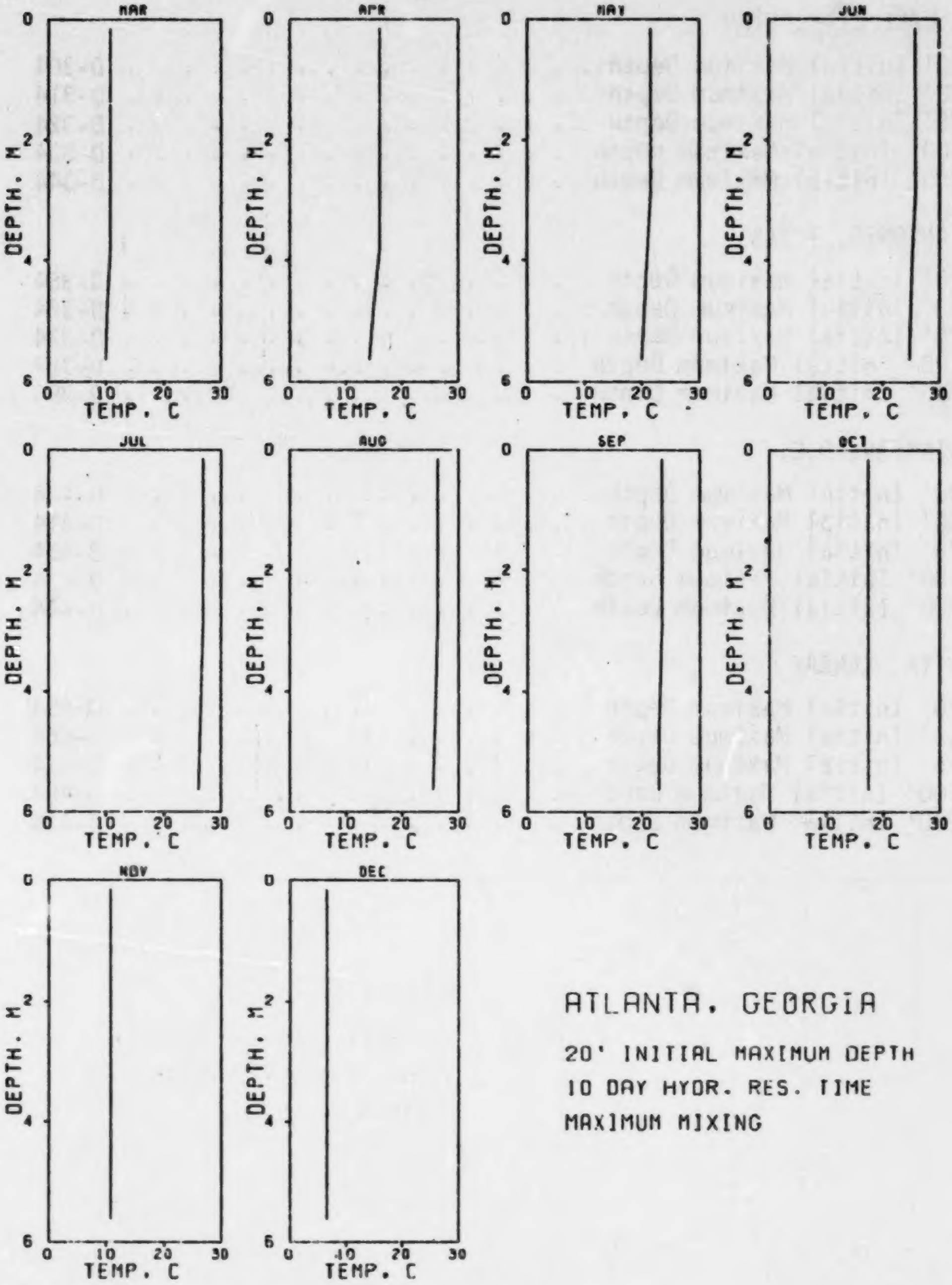
20' Initial Maximum Depth D-354
40' Initial Maximum Depth D-364
75' Initial Maximum Depth D-374
100' Initial Maximum Depth D-384
200' Initial Maximum Depth D-394

WASHINGTON, D.C.

20' Initial Maximum Depth D-404
40' Initial Maximum Depth D-414
75' Initial Maximum Depth D-424
100' Initial Maximum Depth D-434
200' Initial Maximum Depth D-444

WICHITA, KANSAS

20' Initial Maximum Depth D-454
40' Initial Maximum Depth D-464
75' Initial Maximum Depth D-474
100' Initial Maximum Depth D-484
200' Initial Maximum Depth D-494



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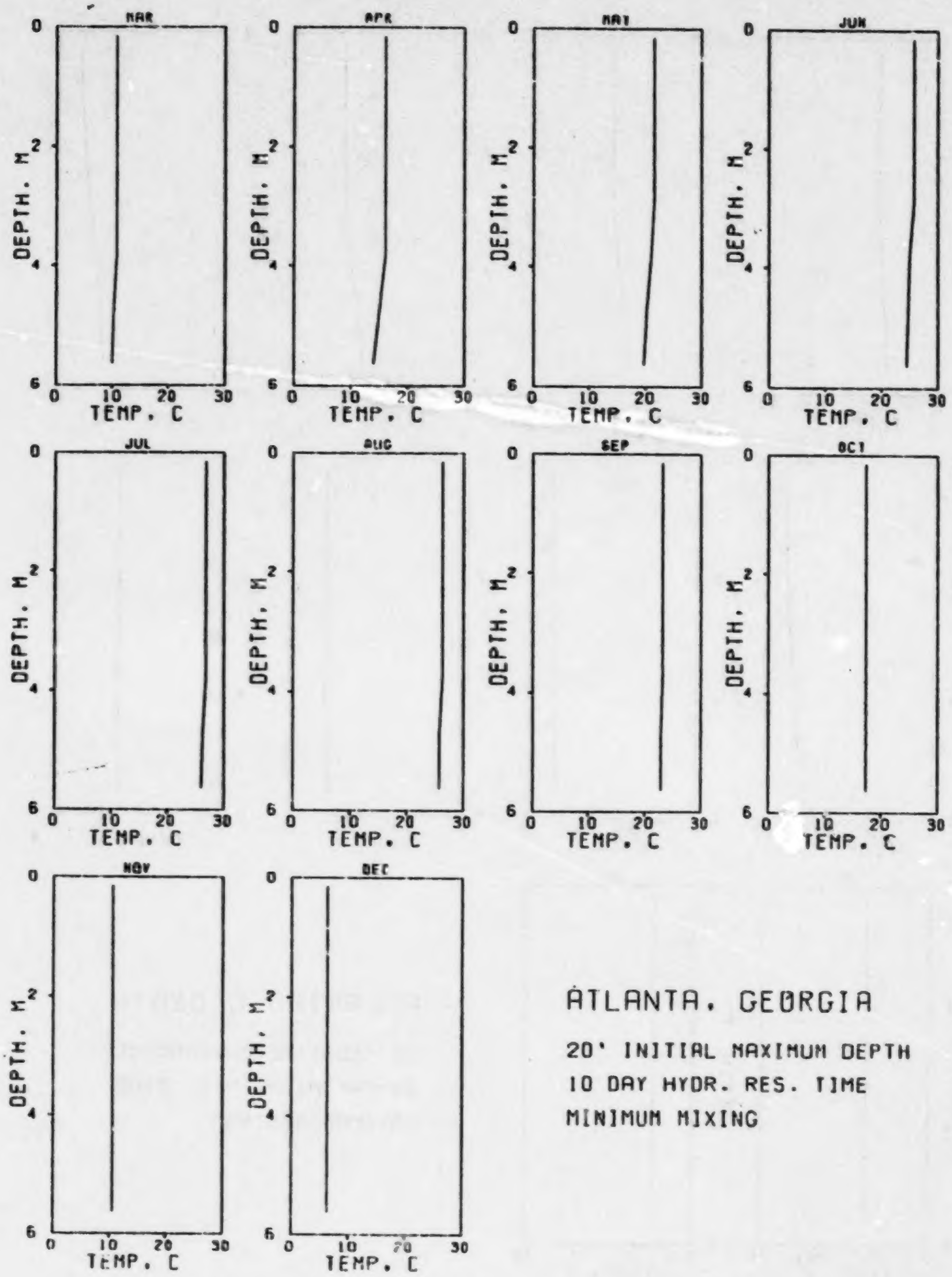
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



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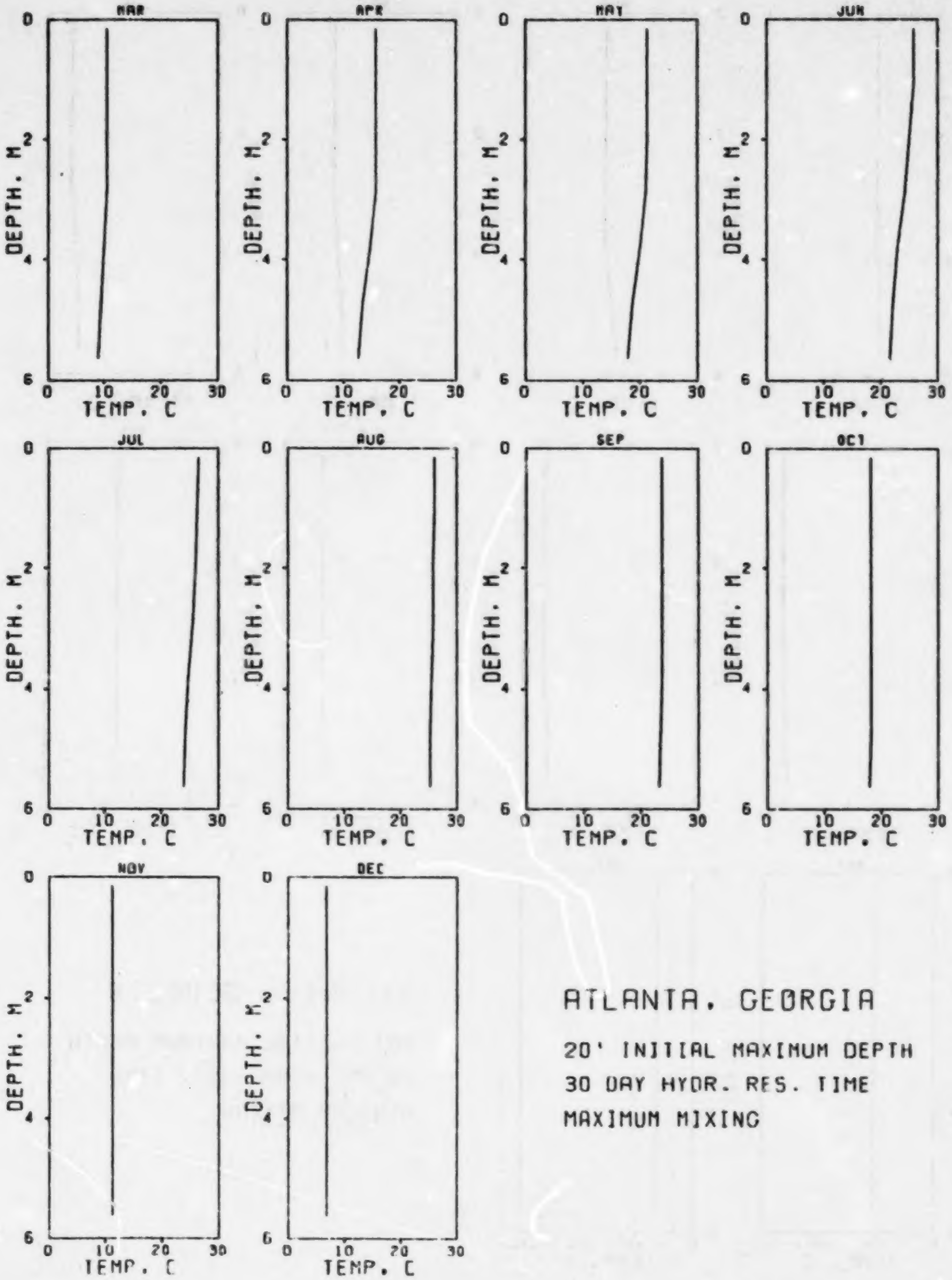

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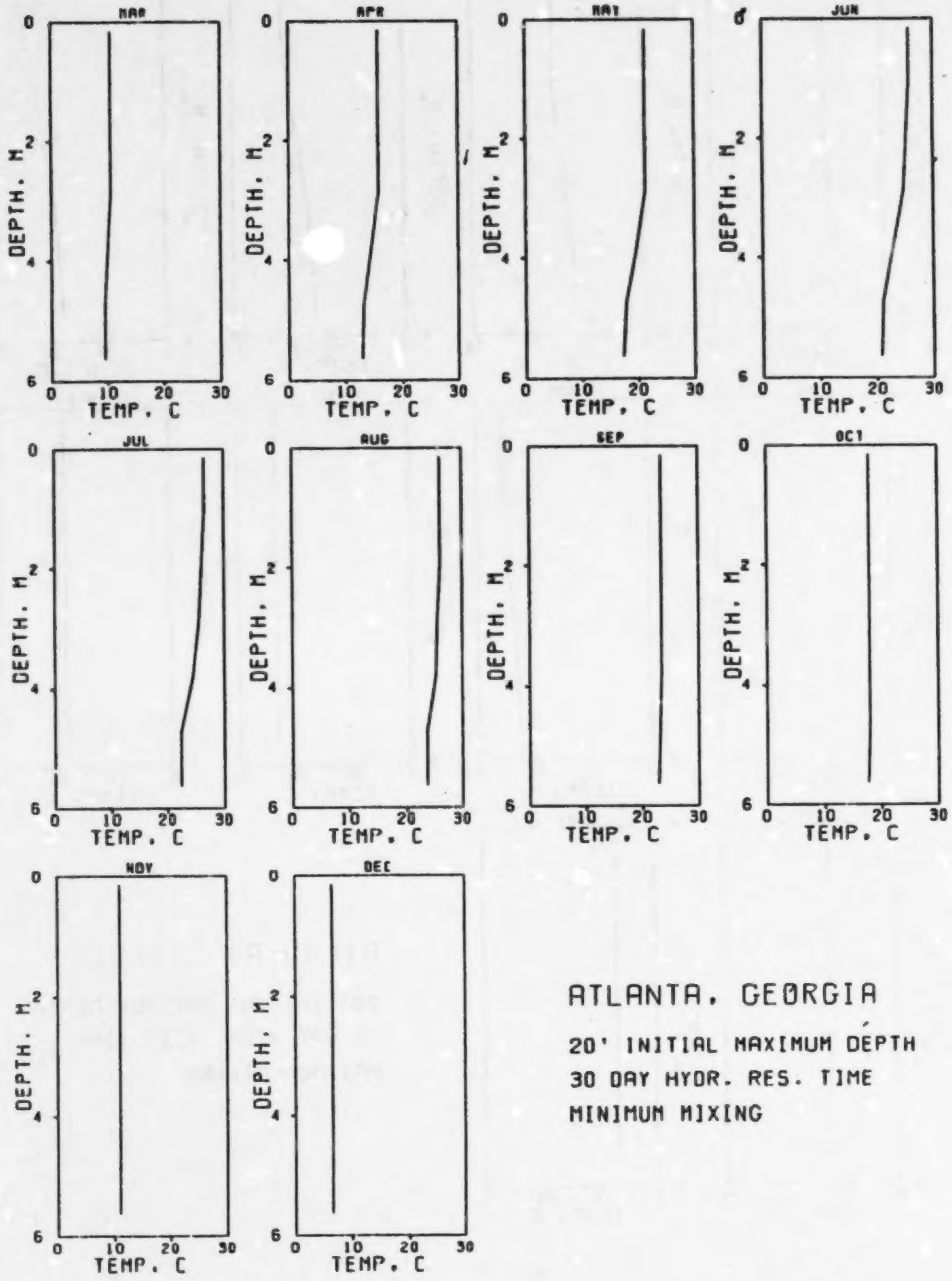

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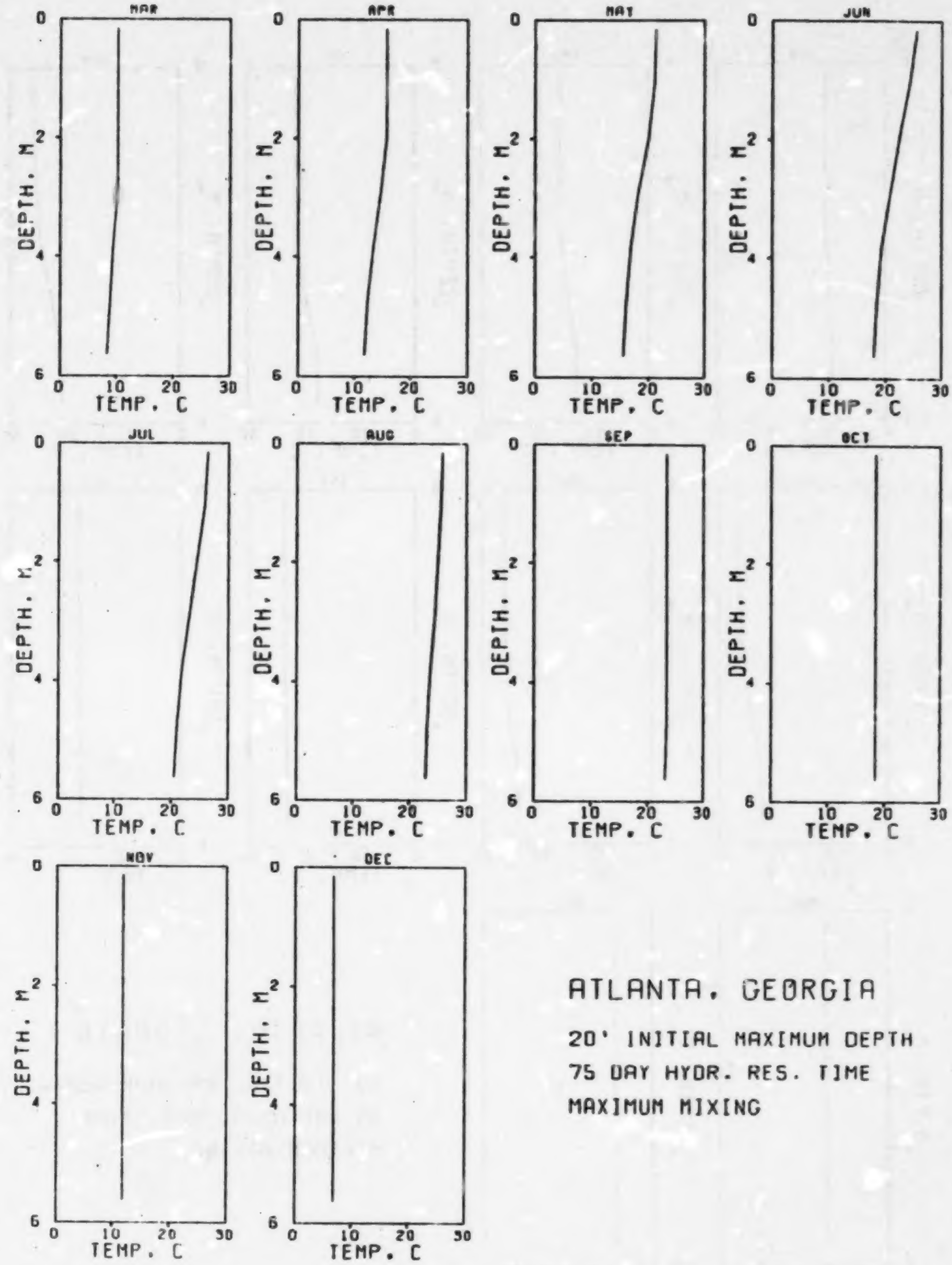

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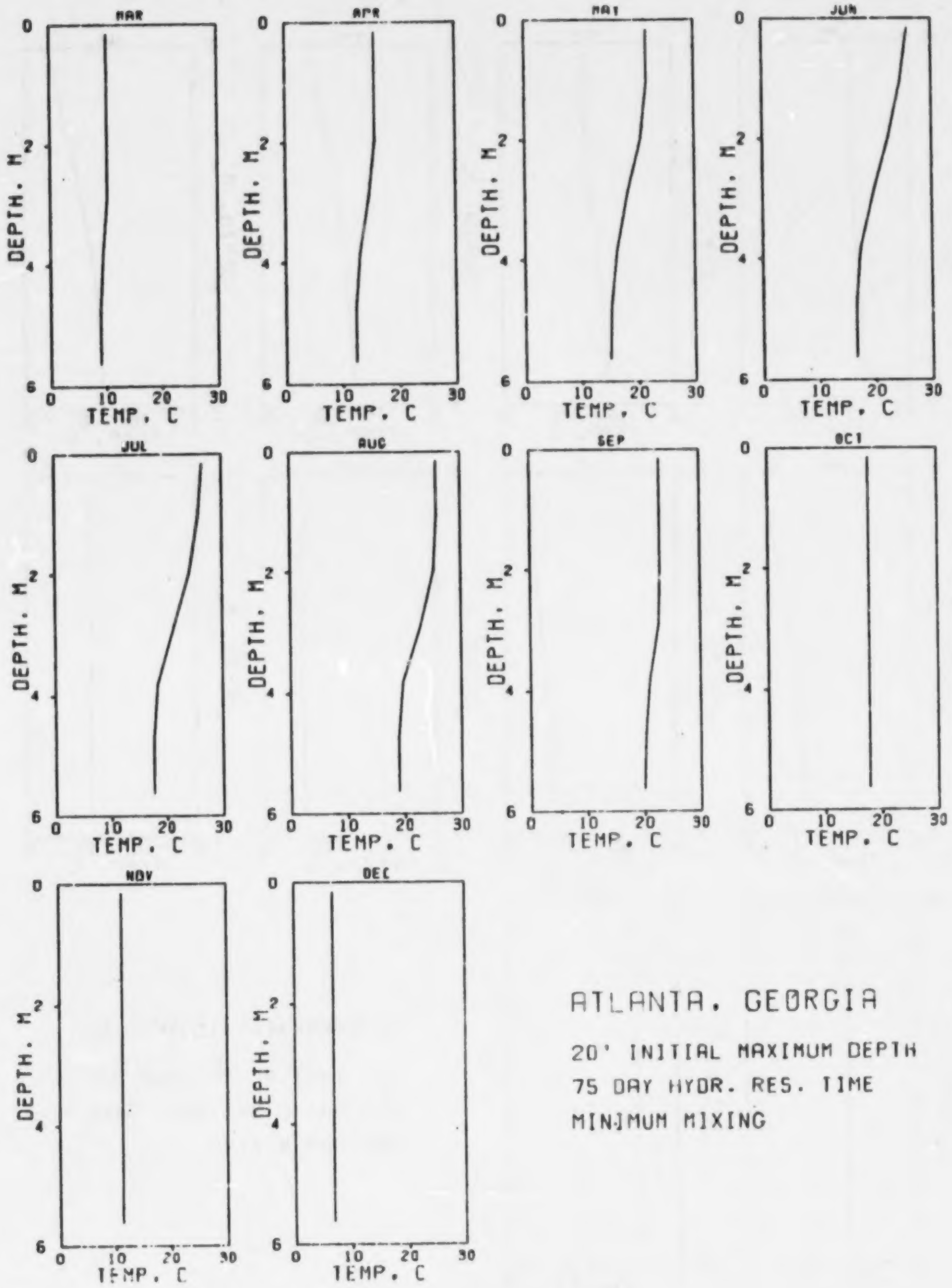

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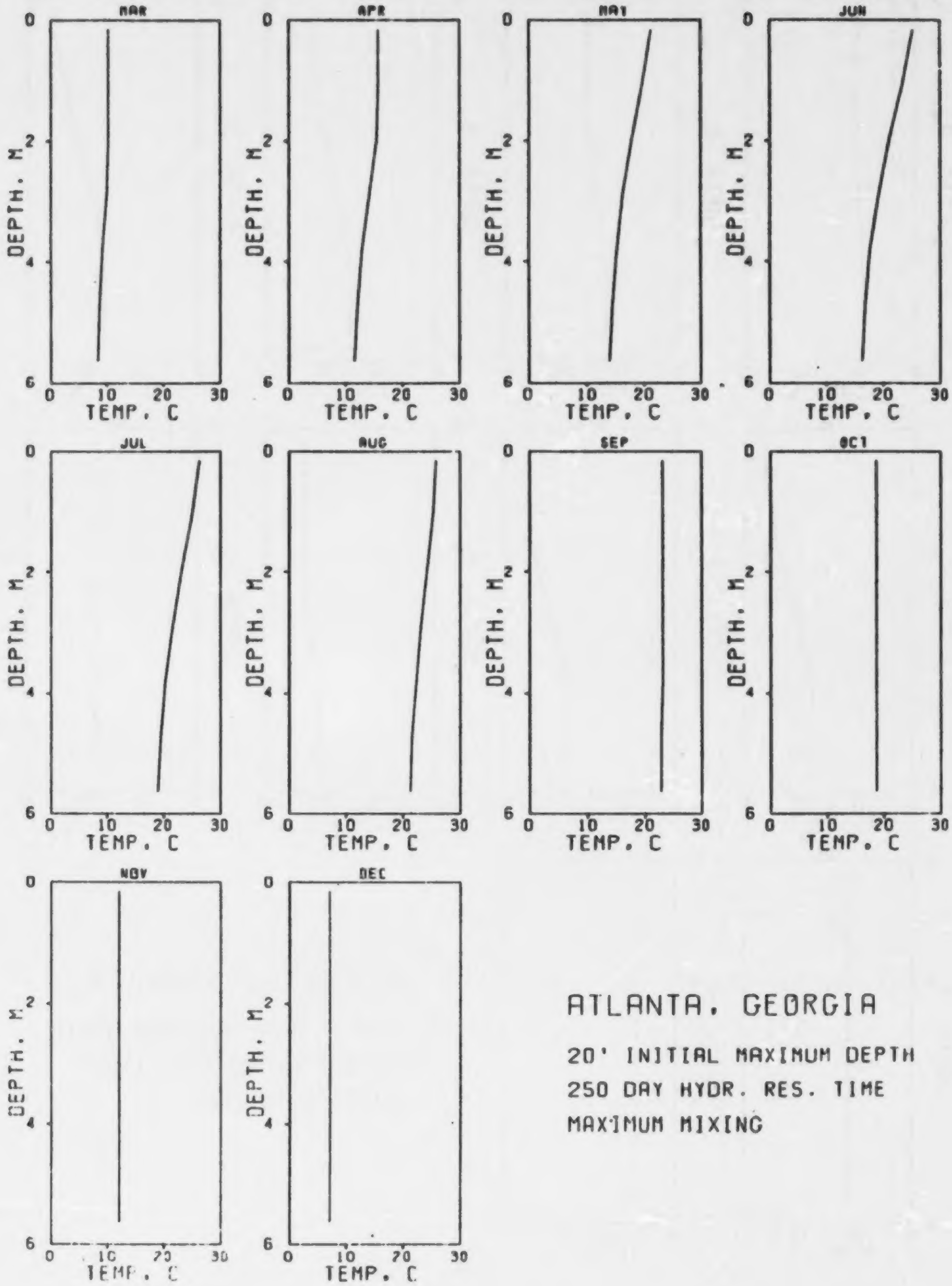

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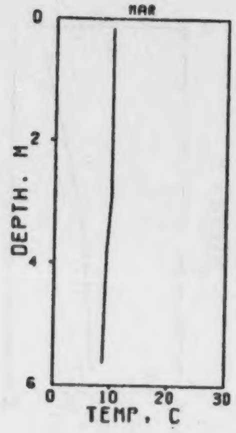
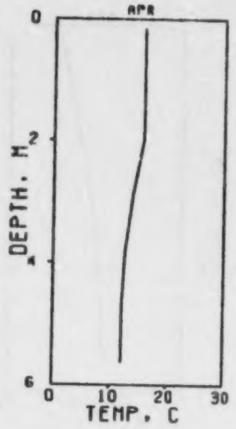
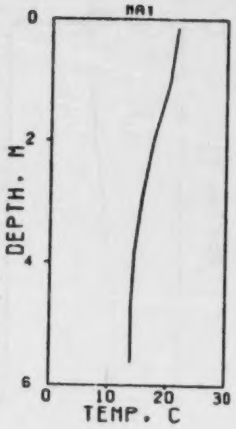
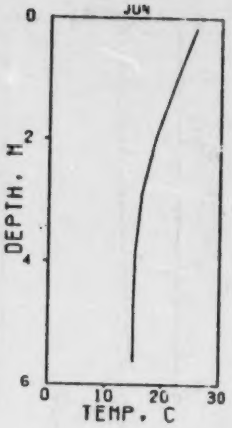
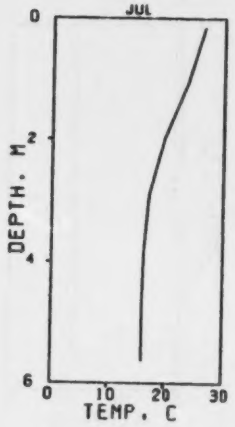
MAXIMUM MIXING

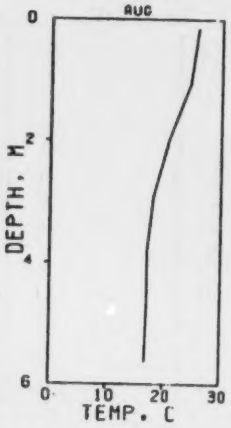


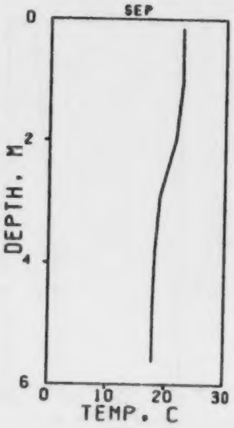


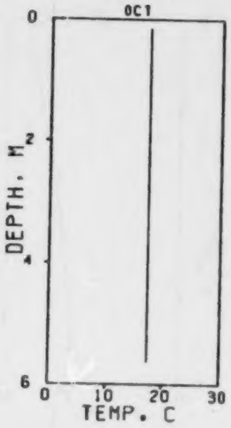


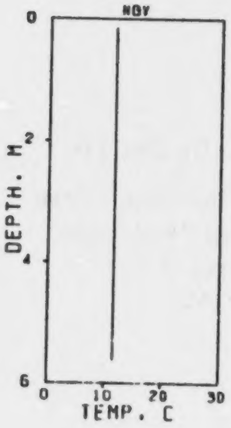


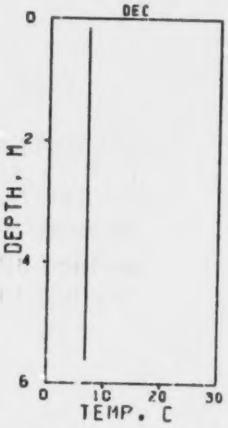












ATLANTA, GEORGIA

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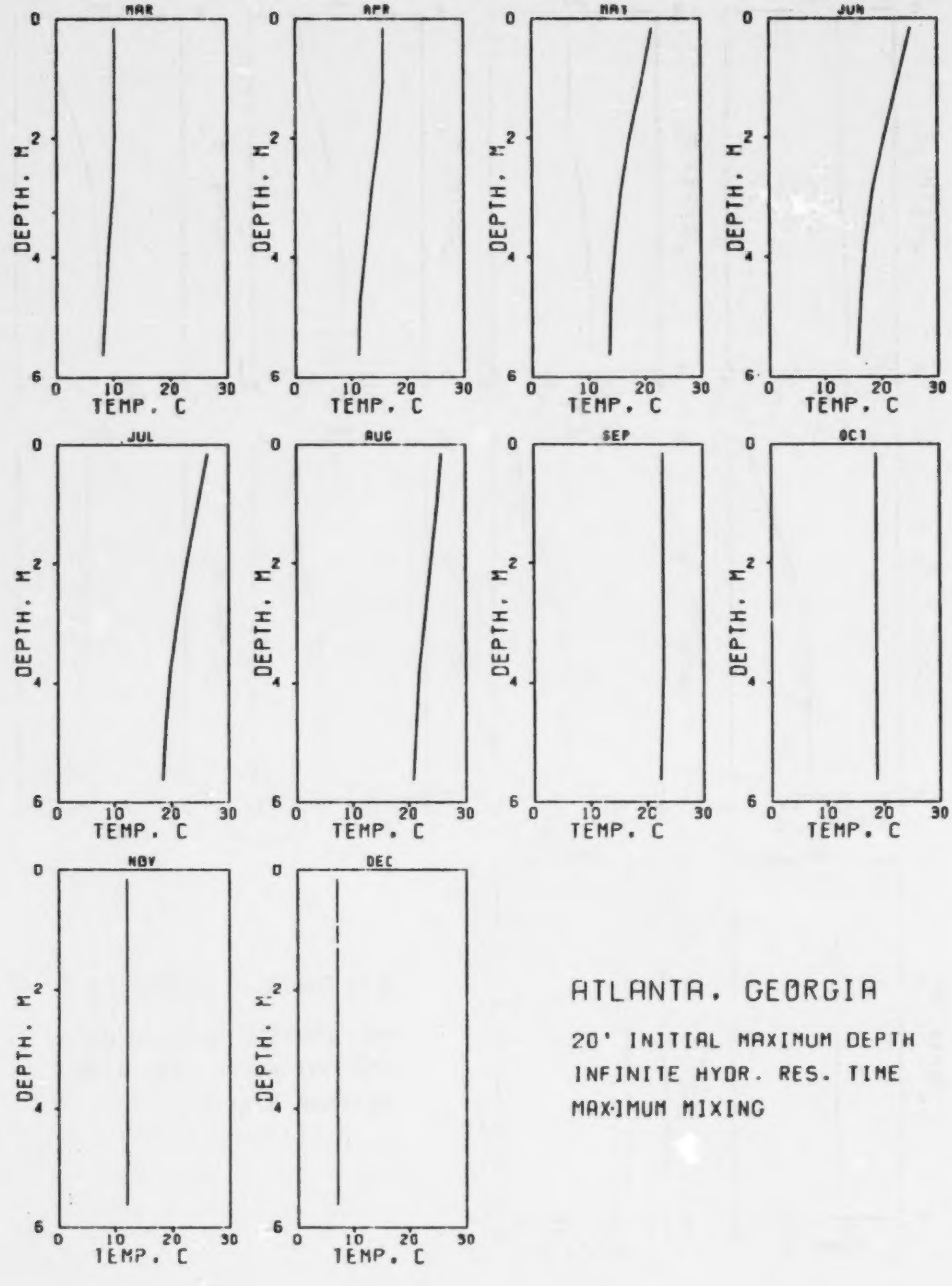
250 DAY HYDR. RES. TIME
MINIMUM MIXING



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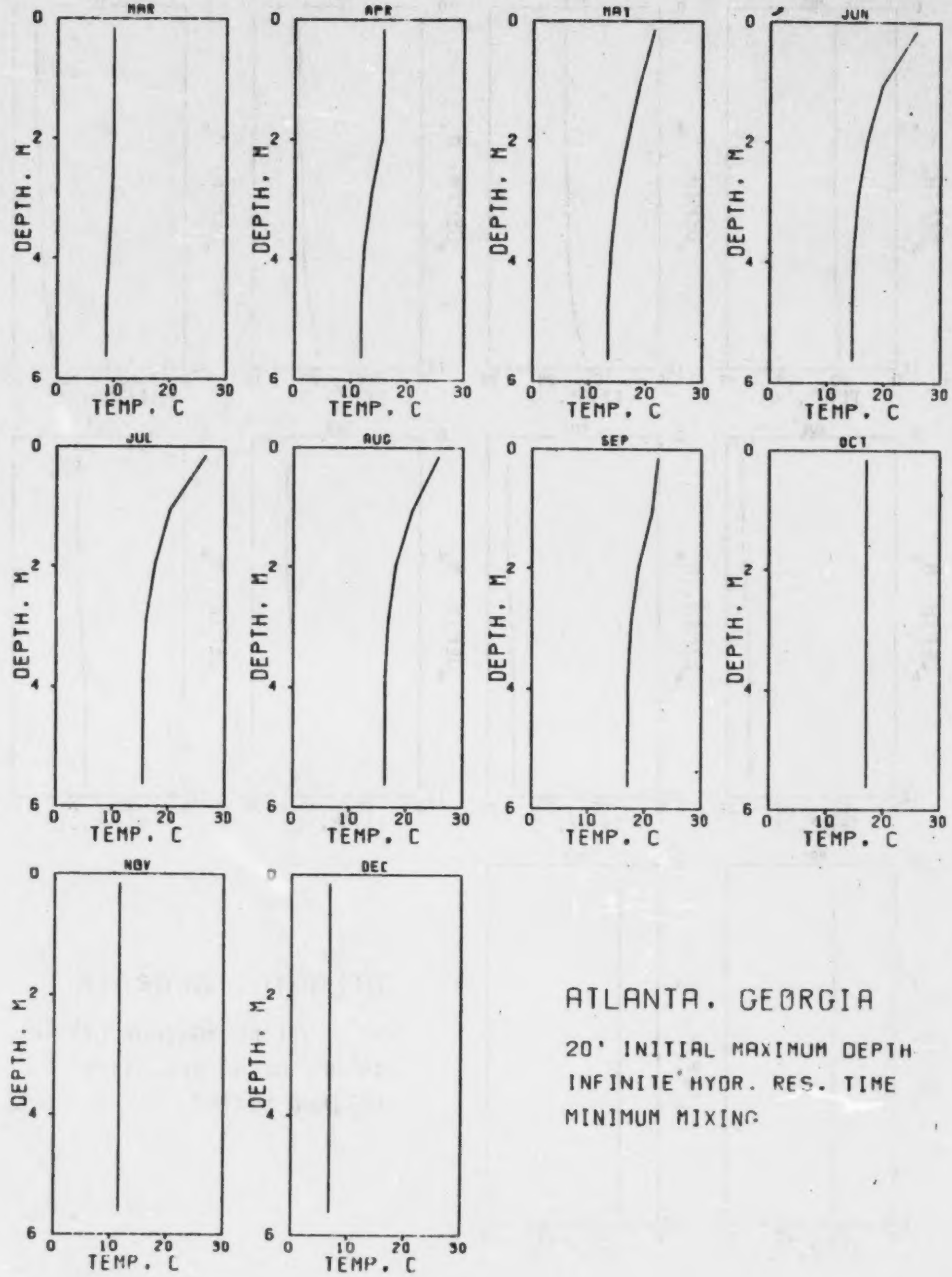

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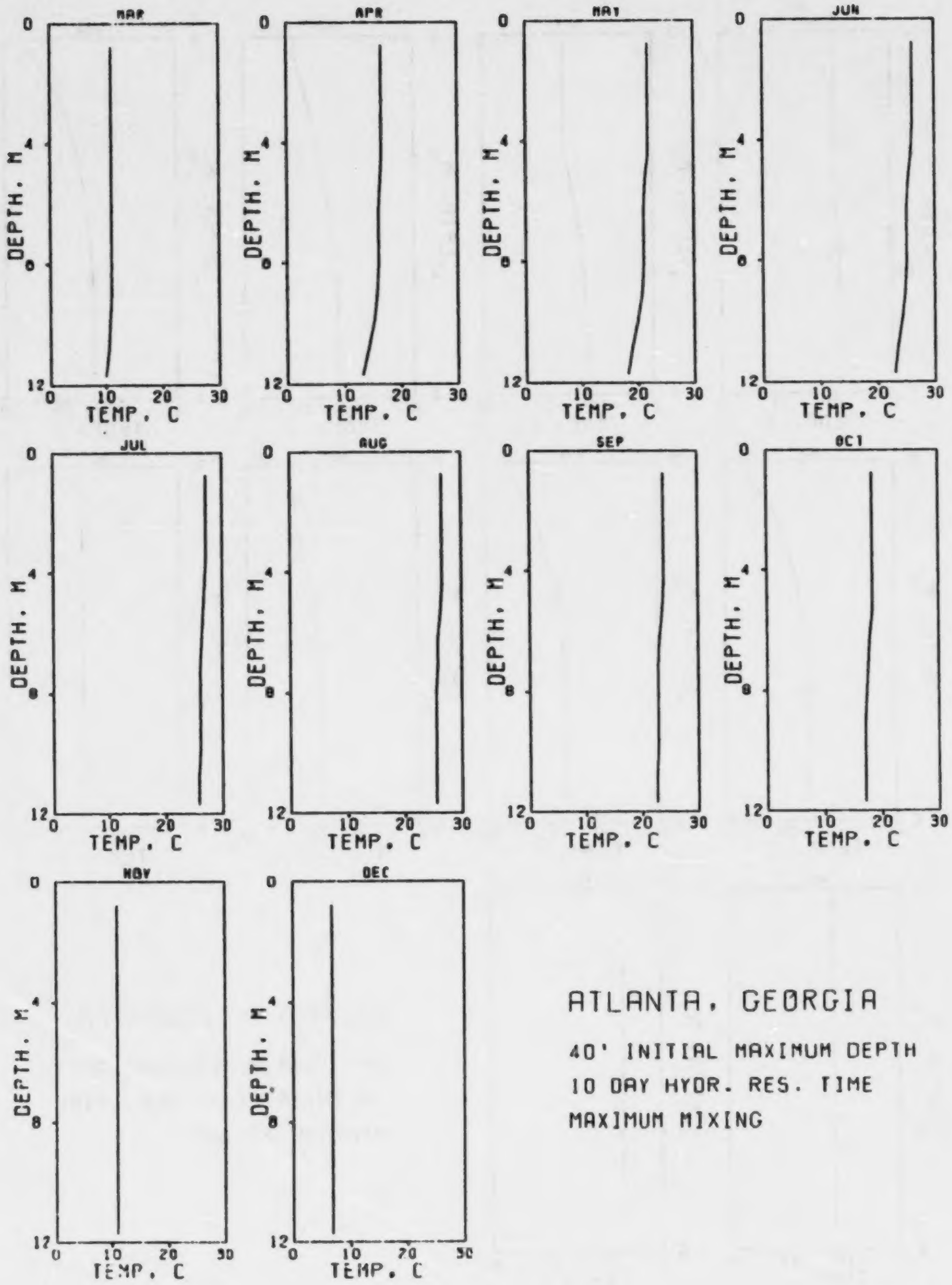

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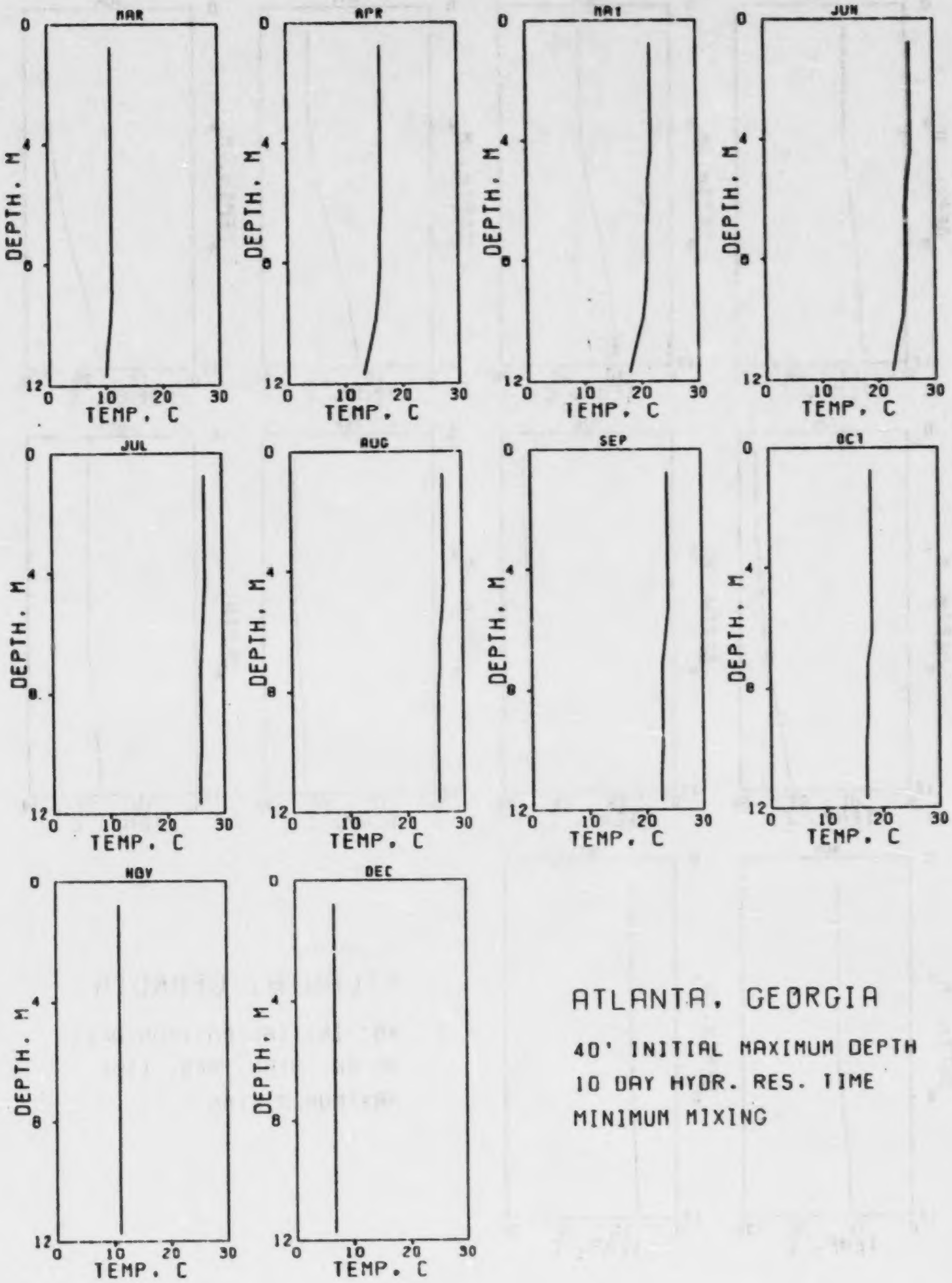

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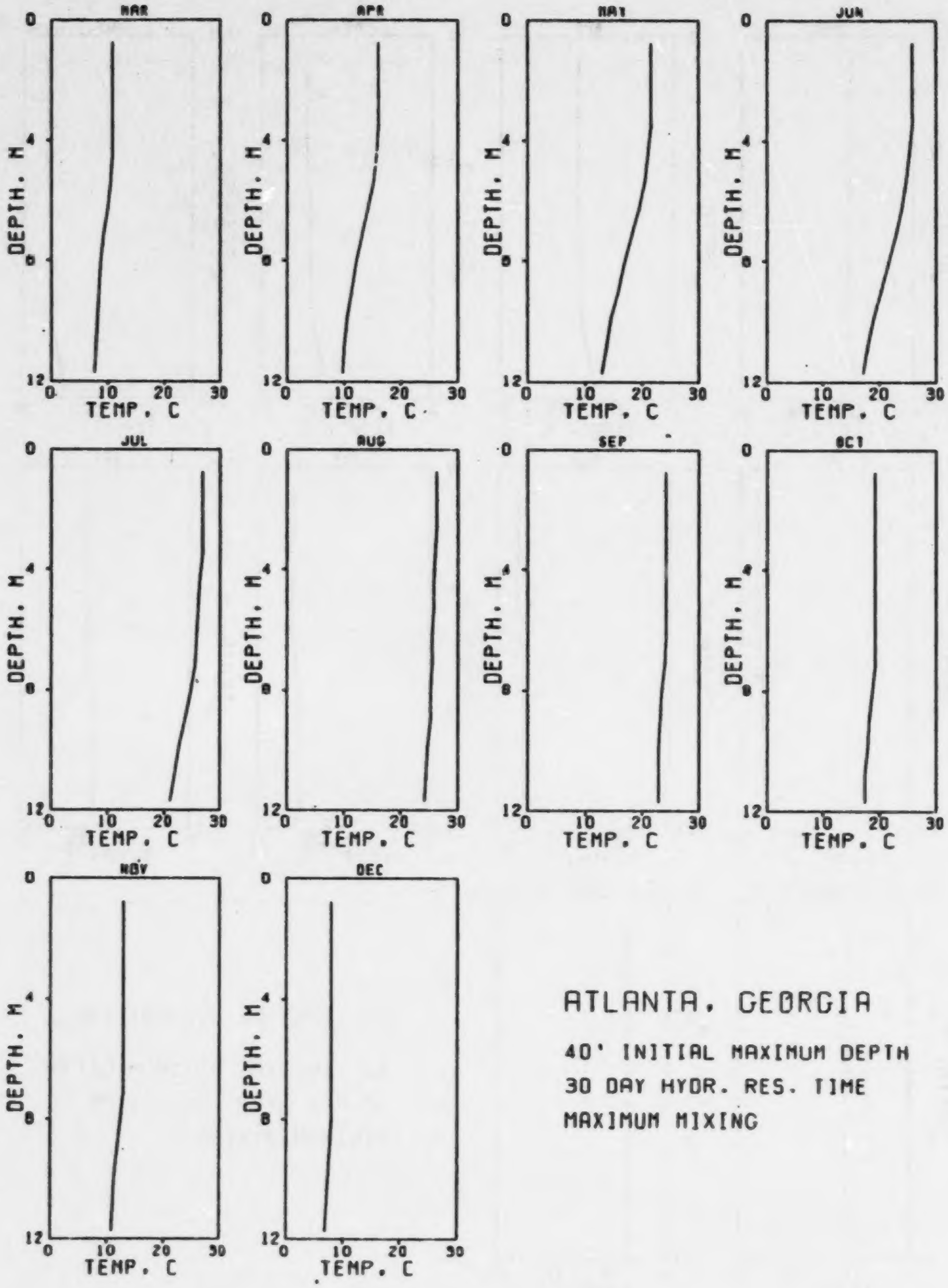

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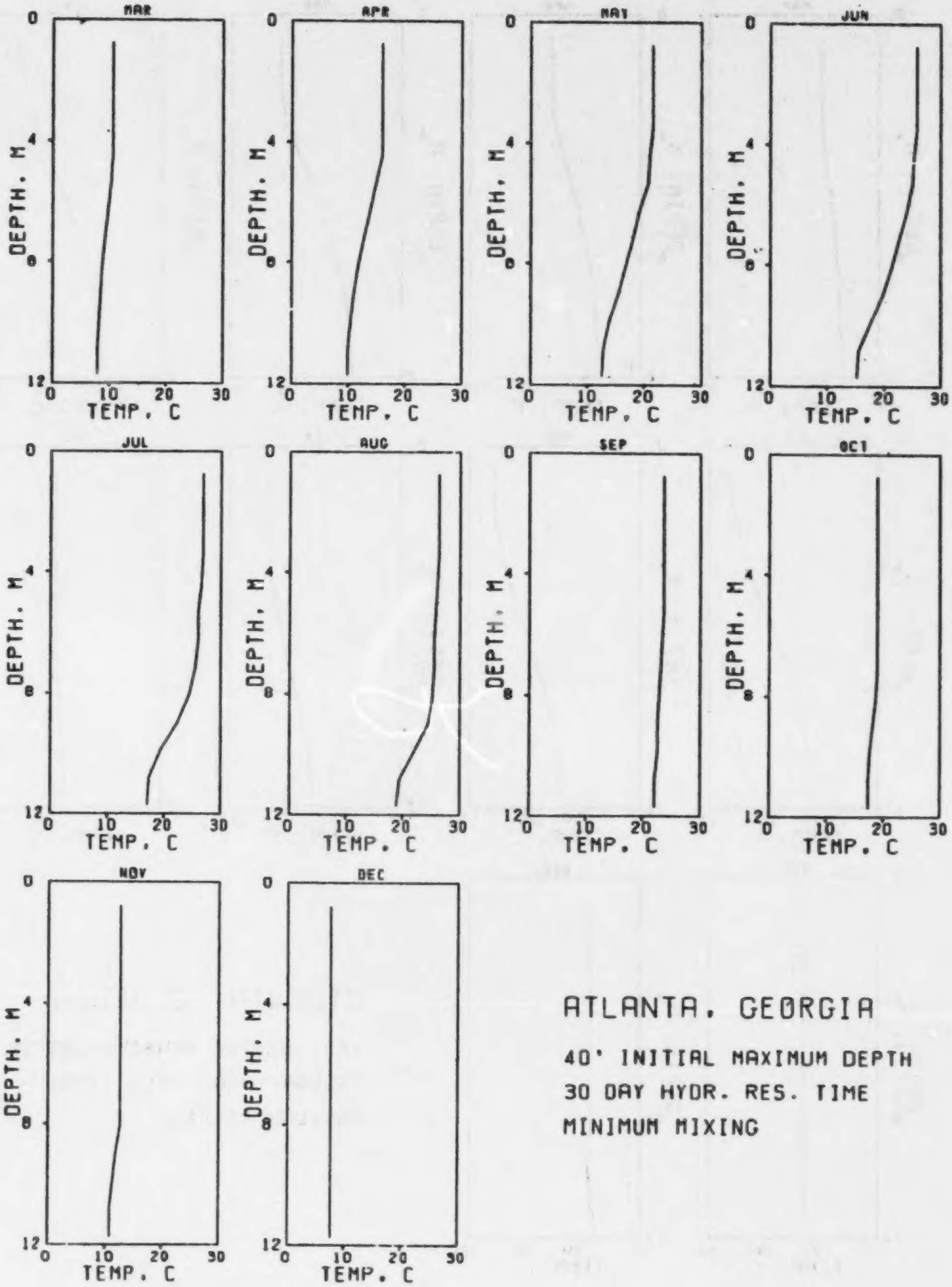

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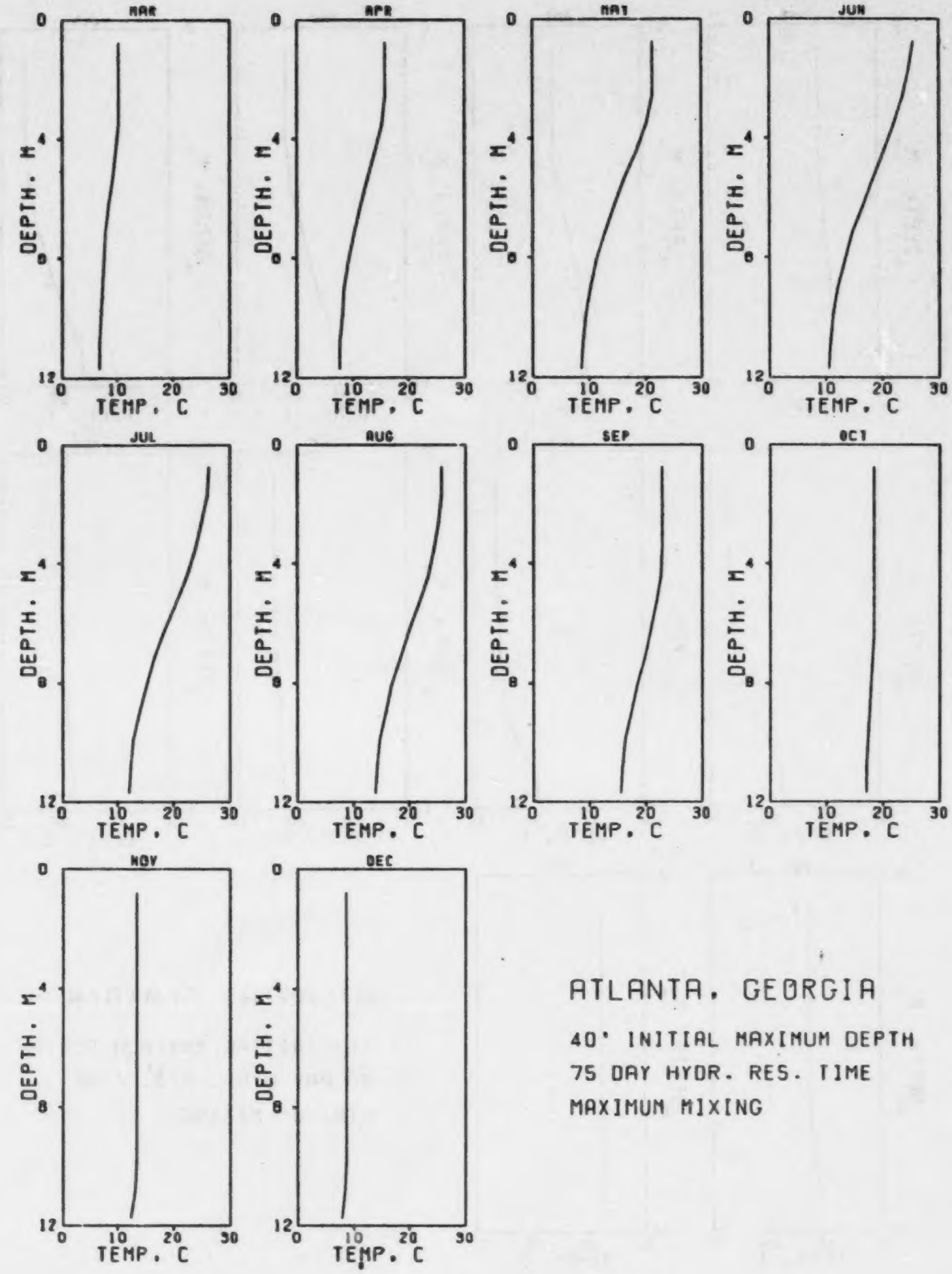

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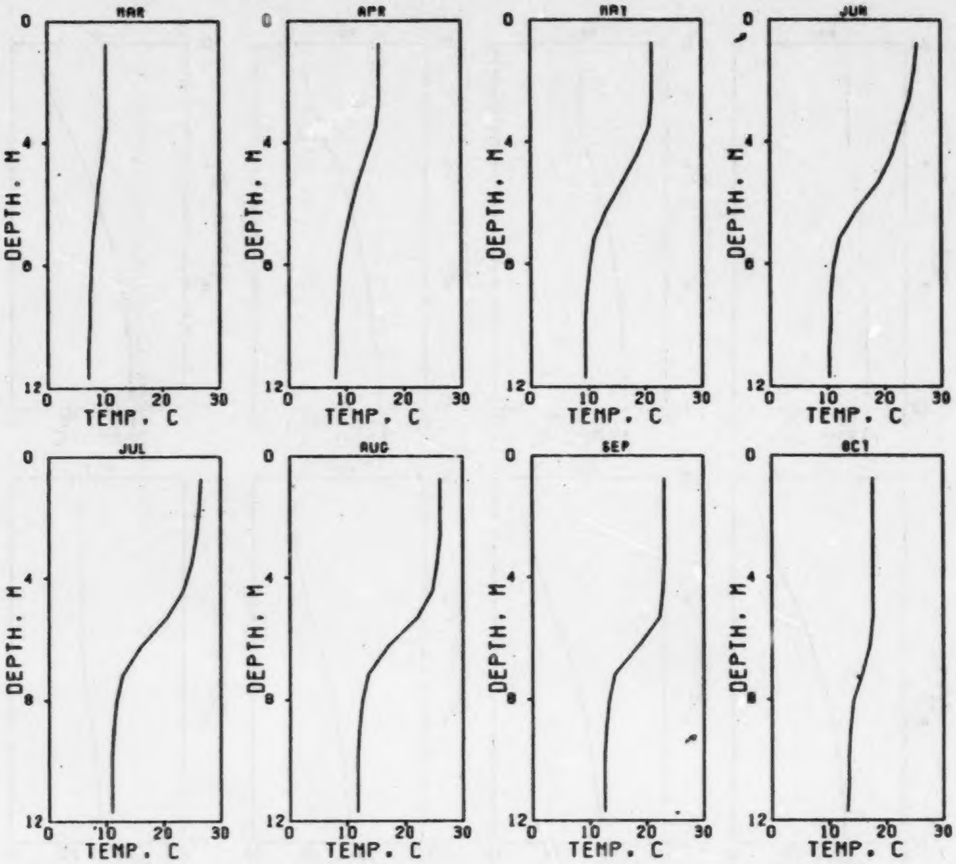

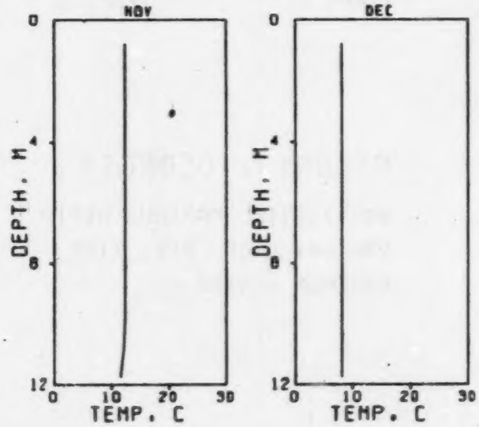

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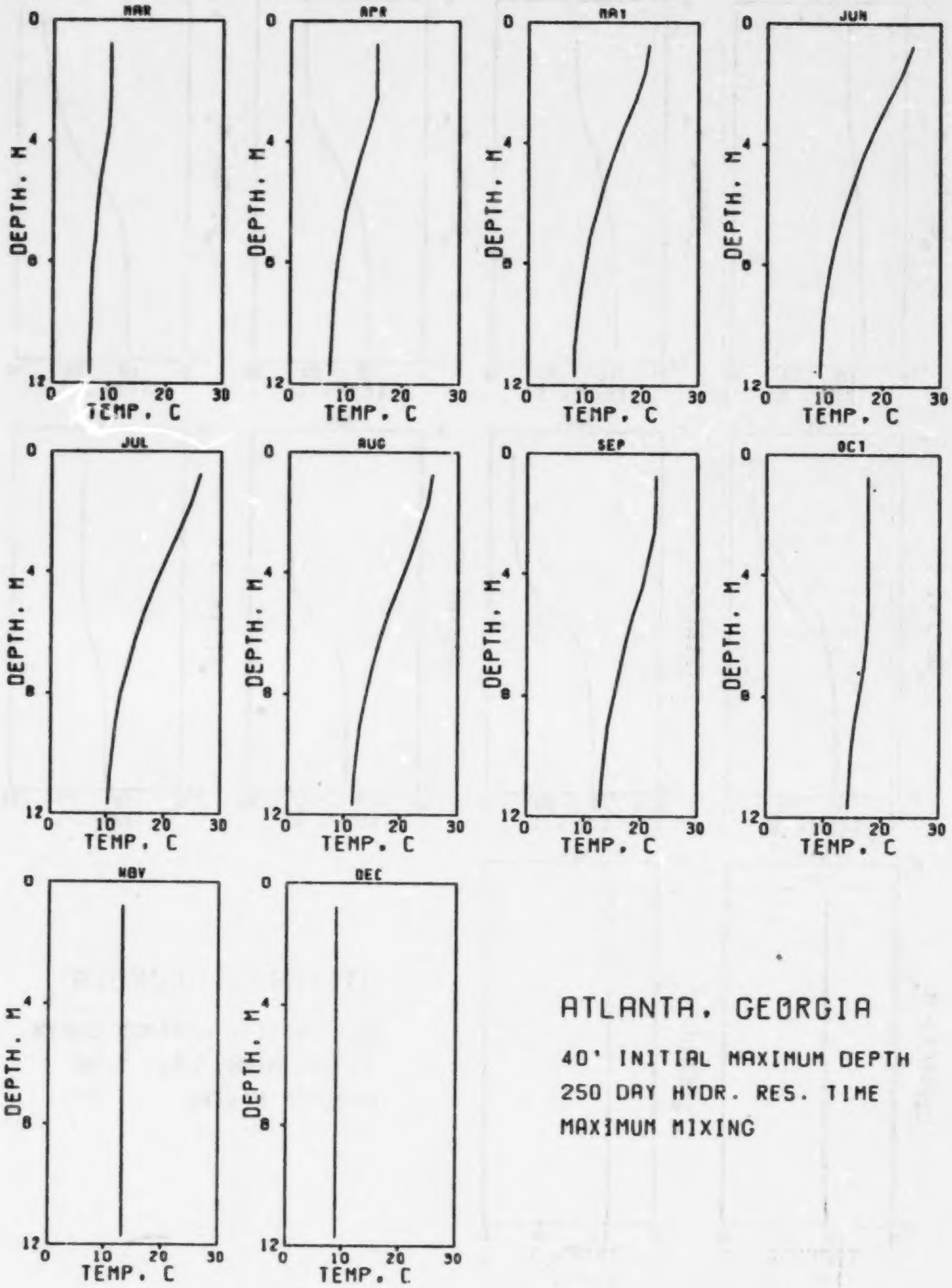

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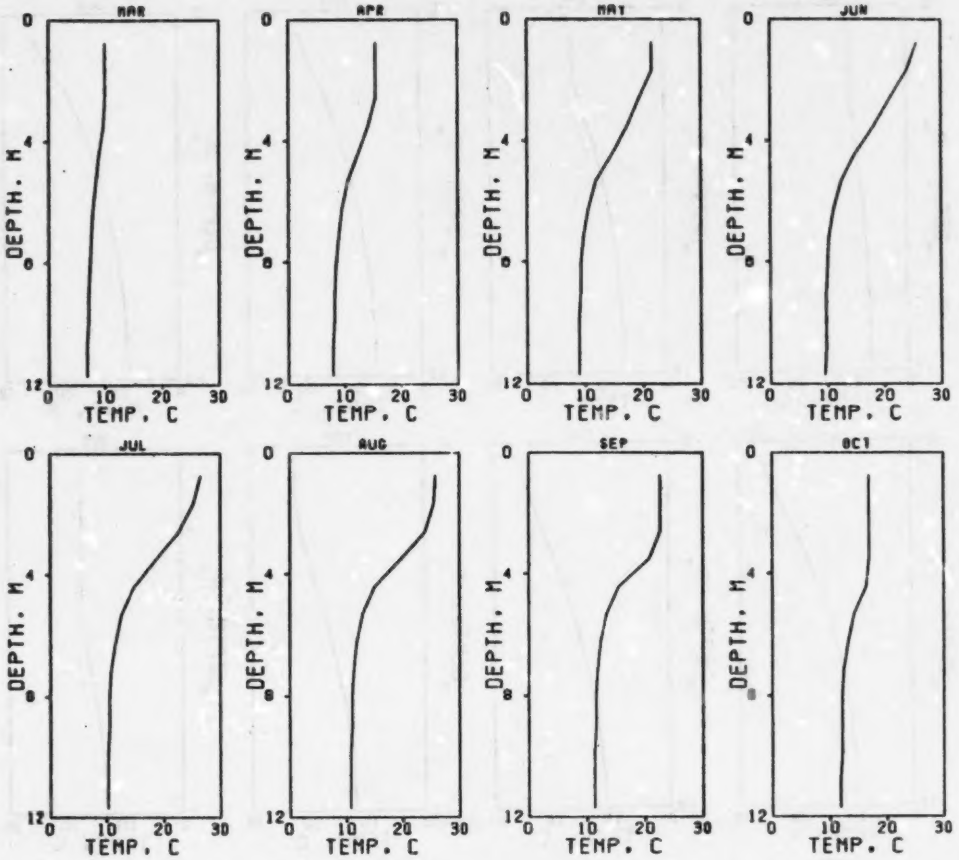
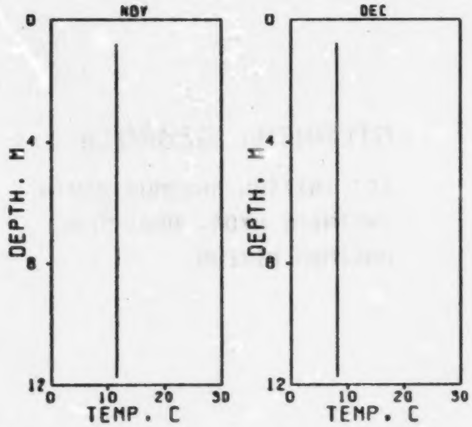
MAXIMUM MIXING





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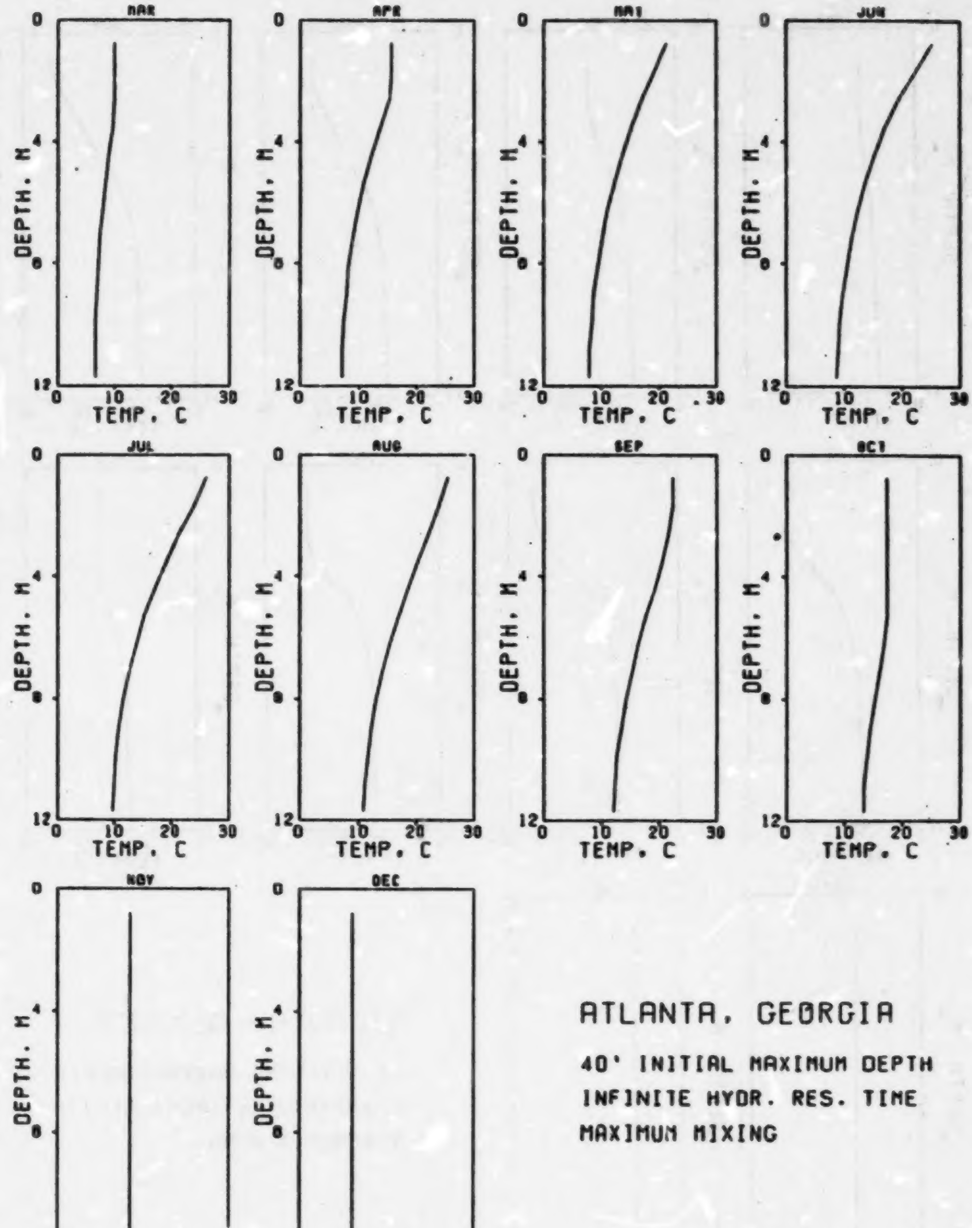

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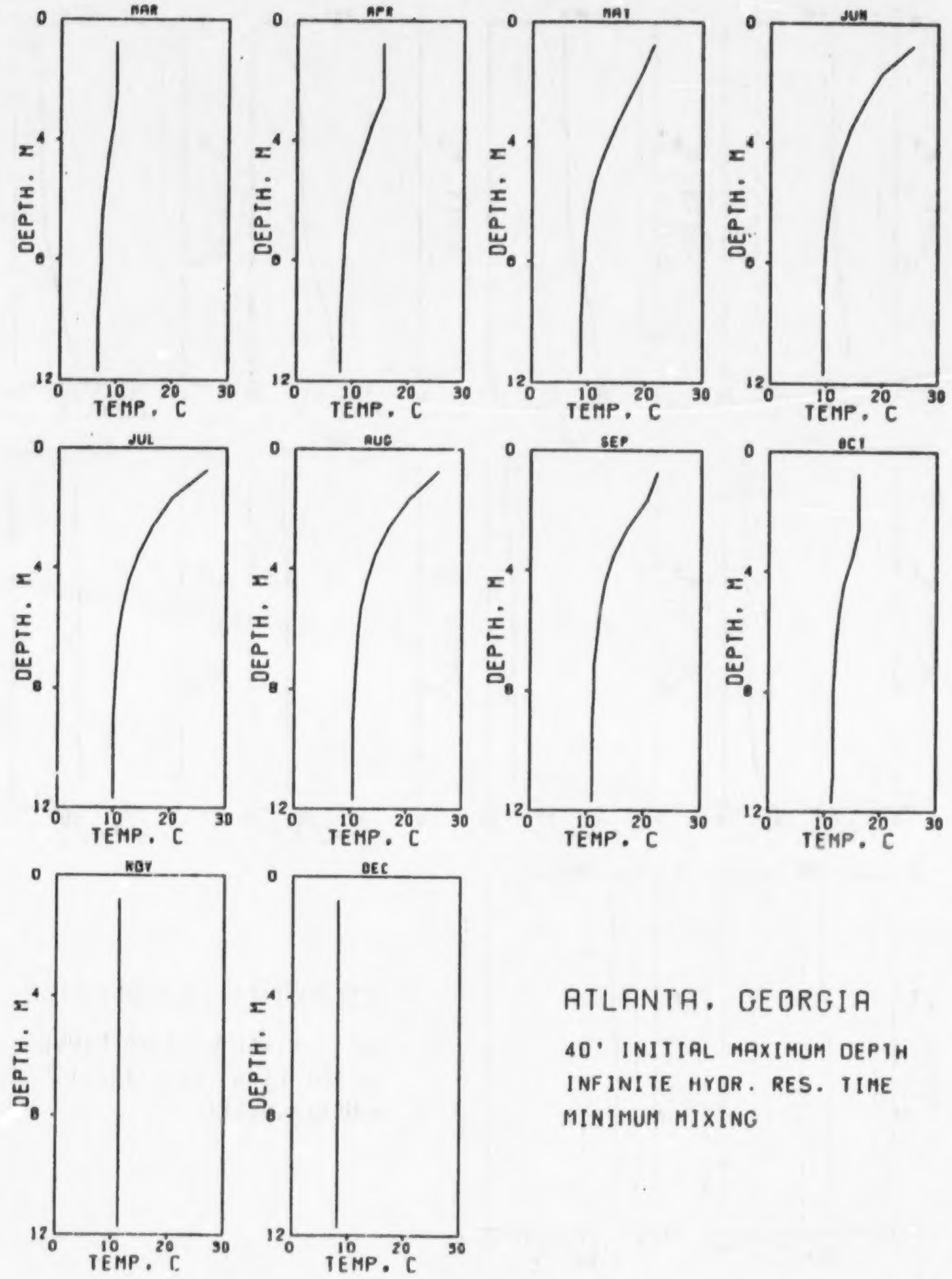
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



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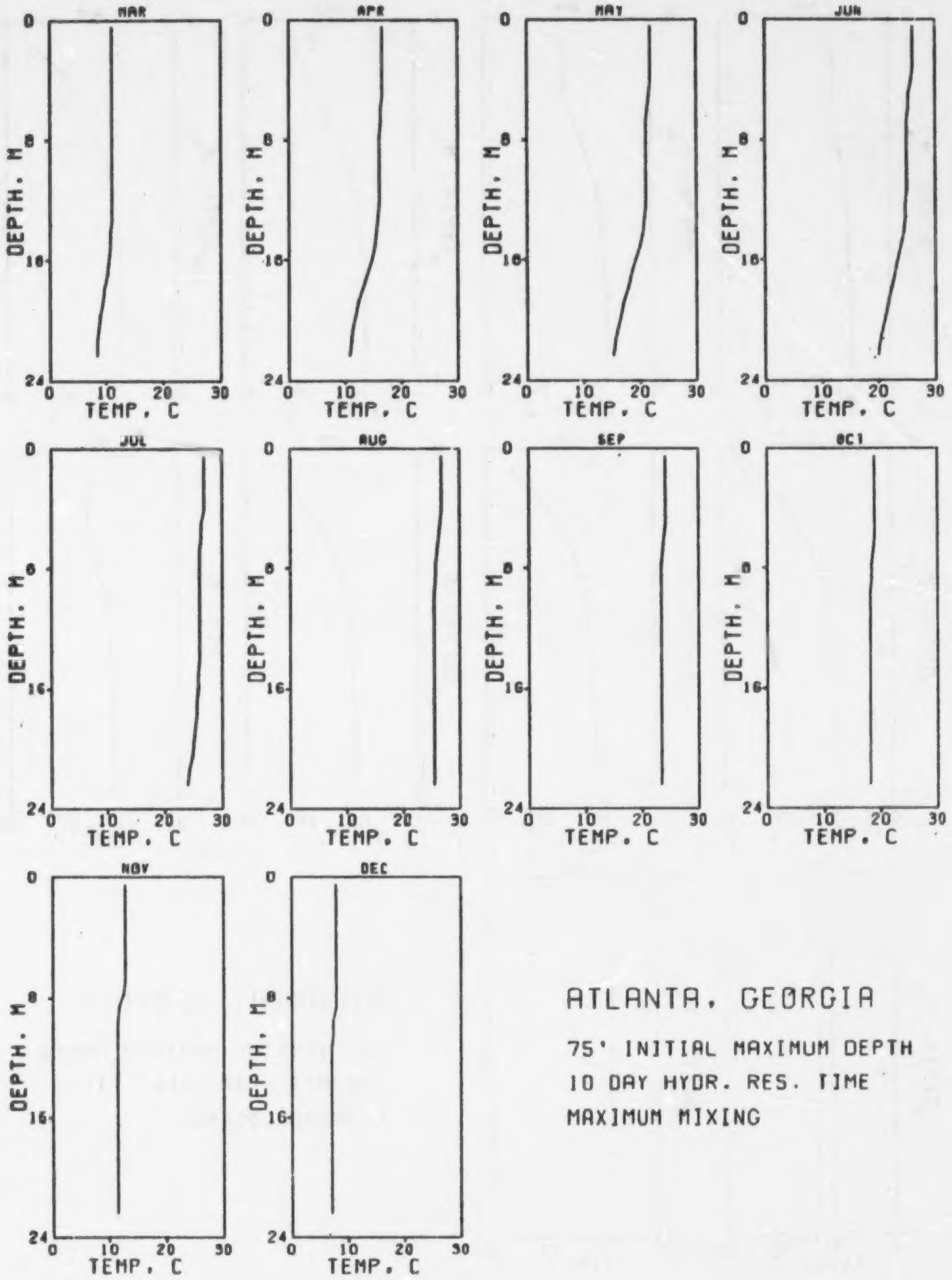

ATLANTA, GEORGIA

75' INITIAL MAXIMUM DEPTH
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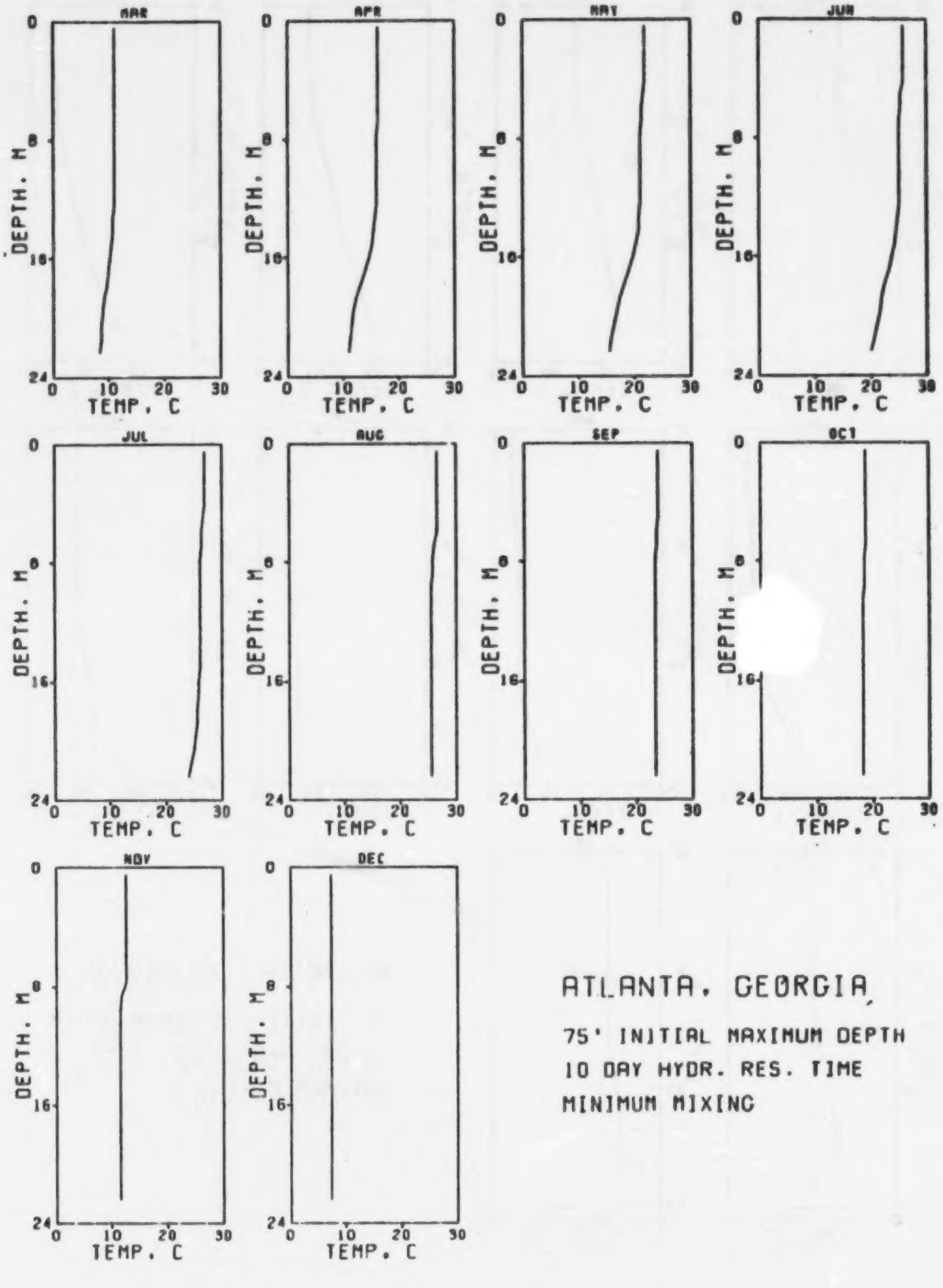

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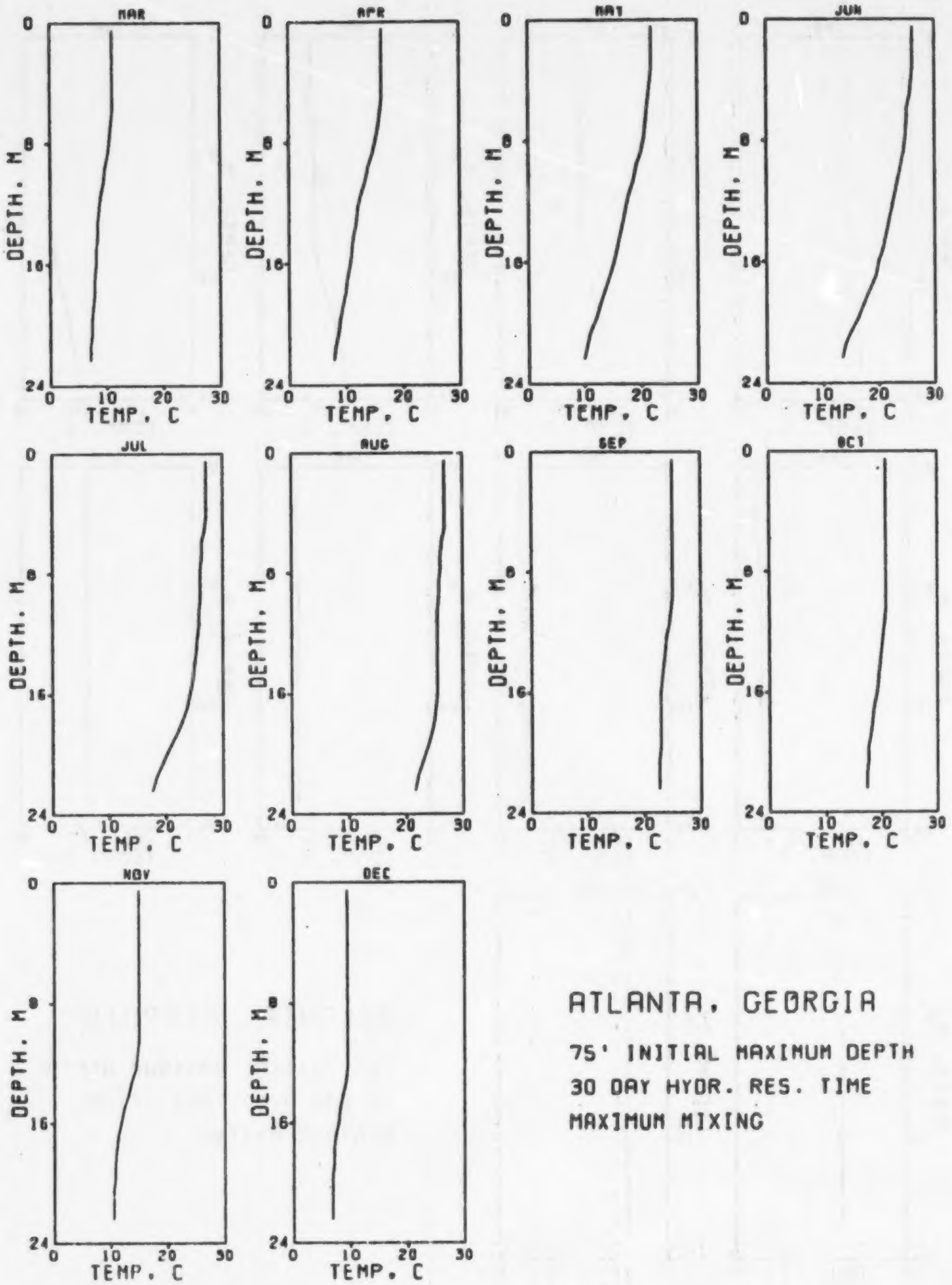

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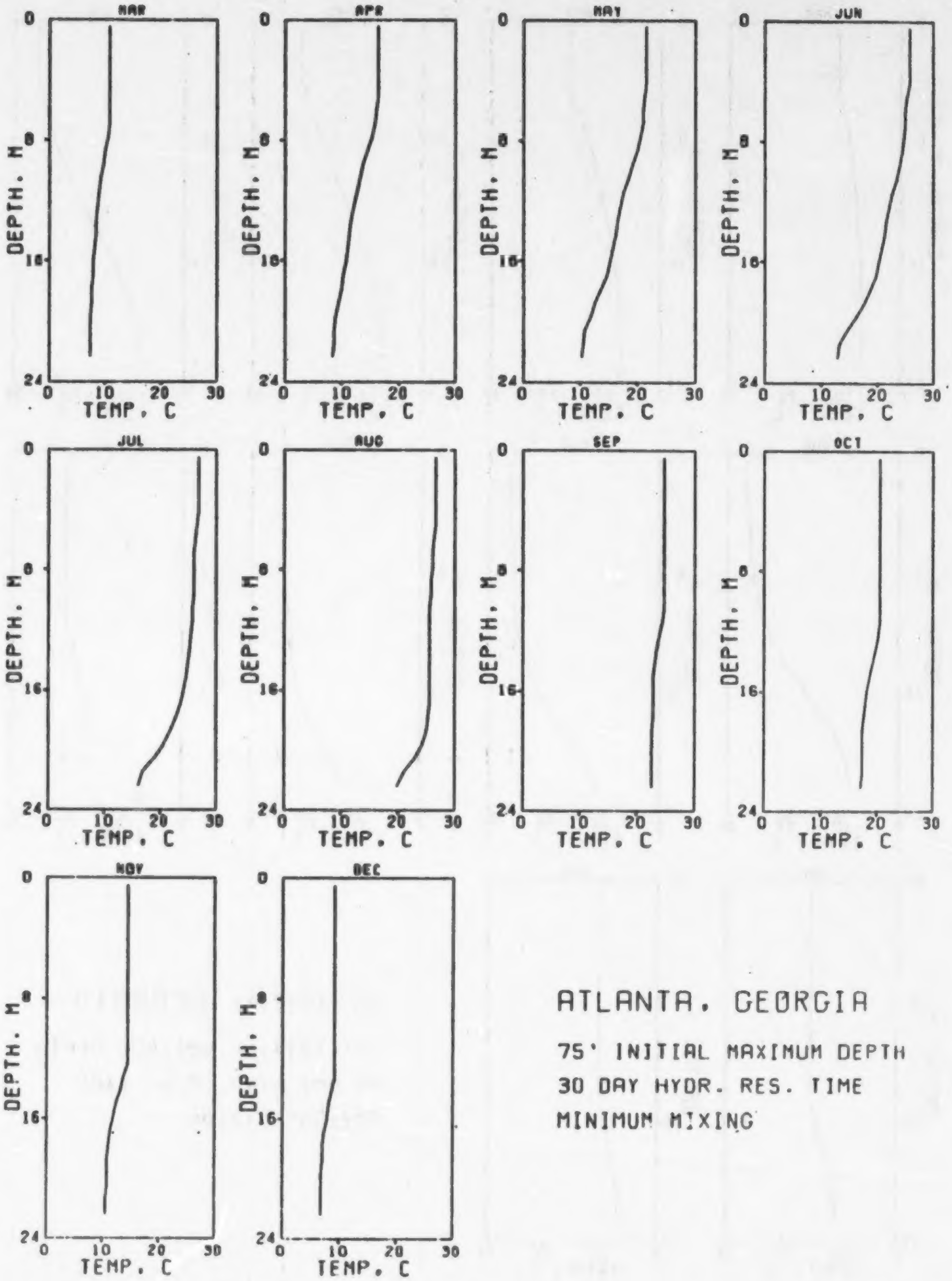

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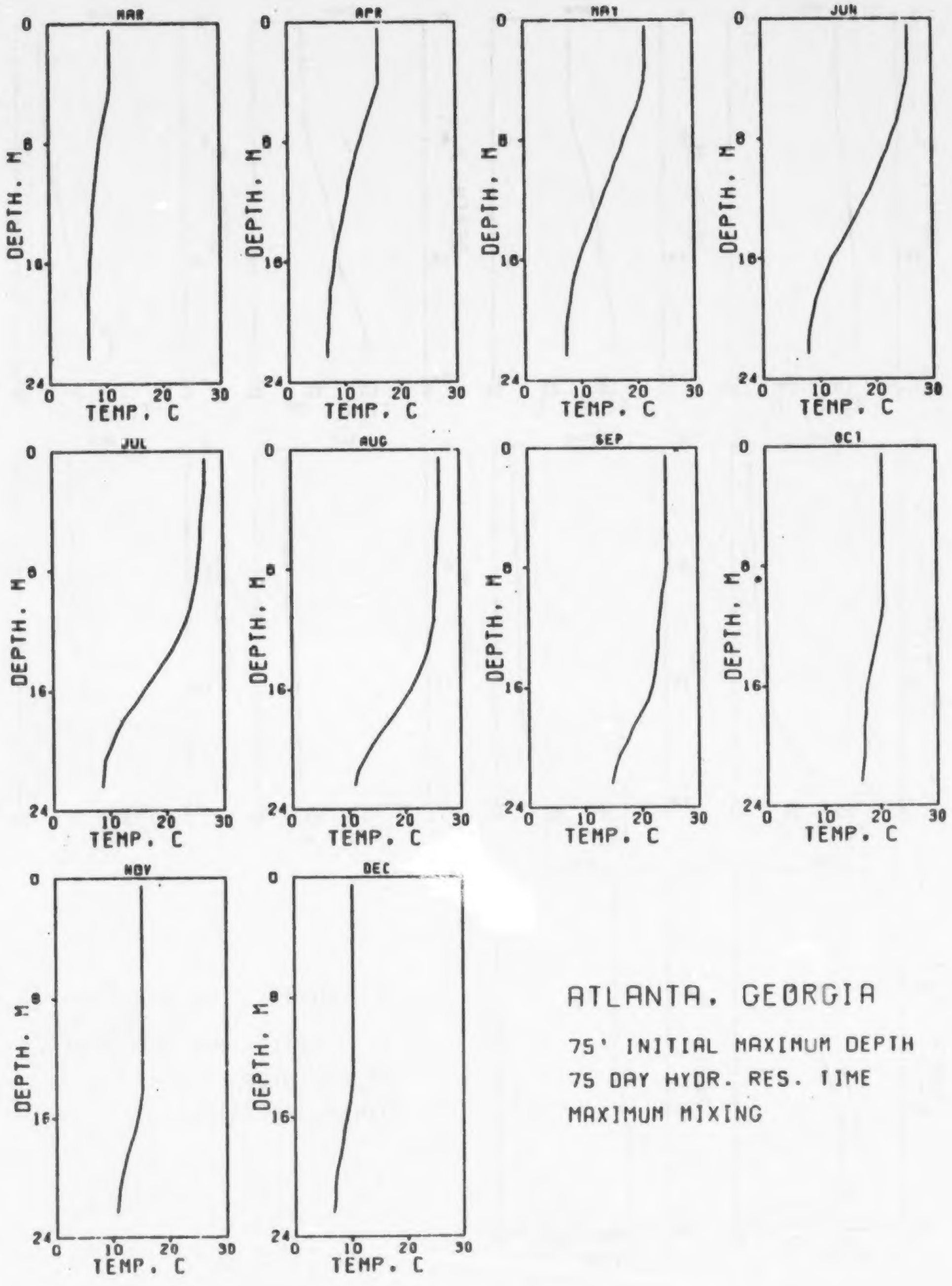

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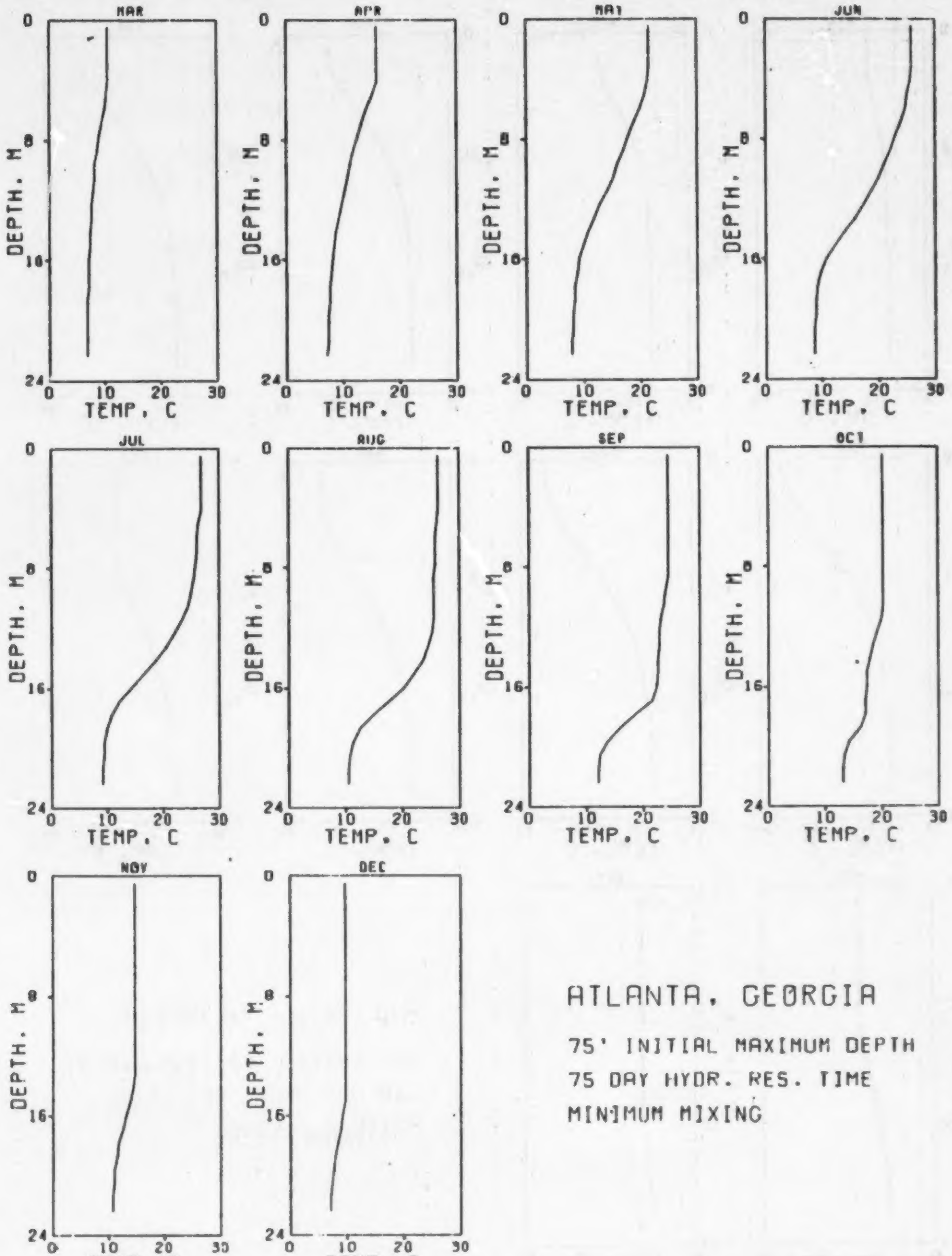

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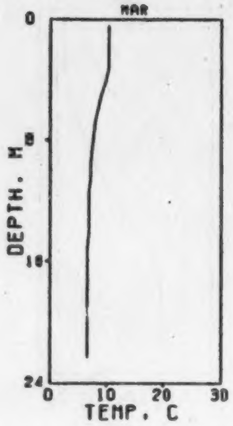
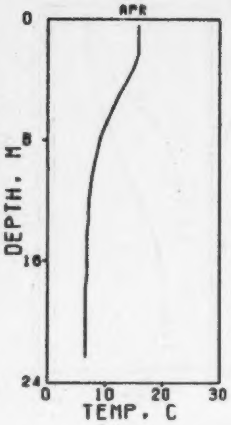
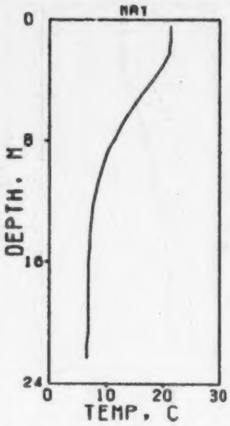
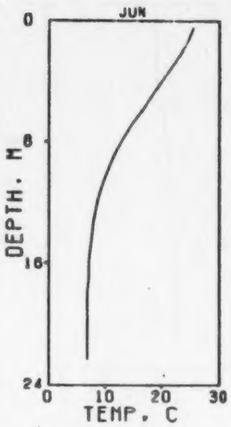
75 DAY HYDR. RES. TIME

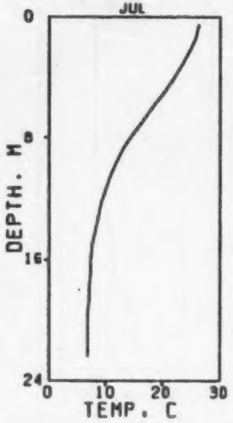
MINIMUM MIXING

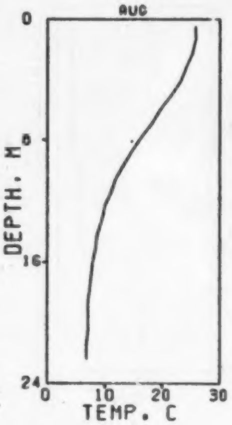


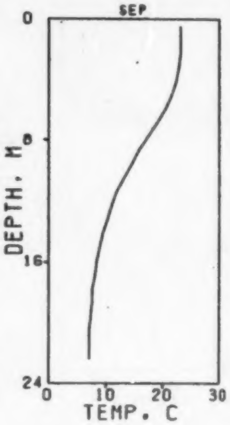


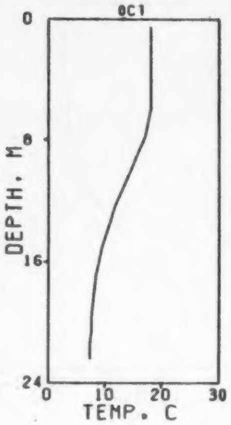


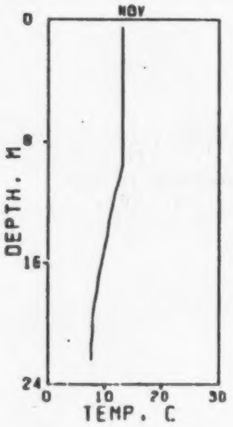


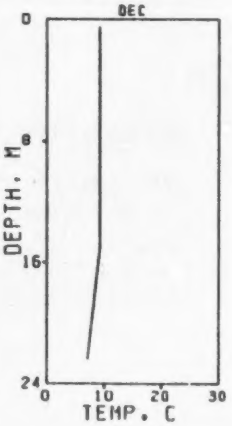






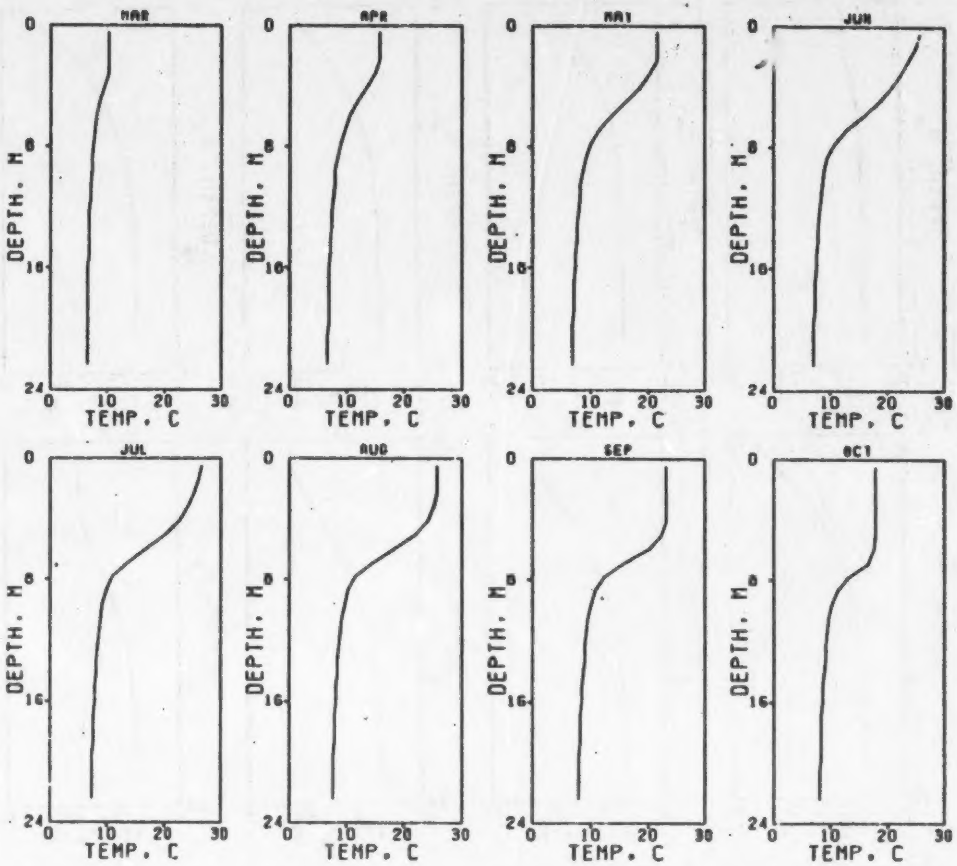


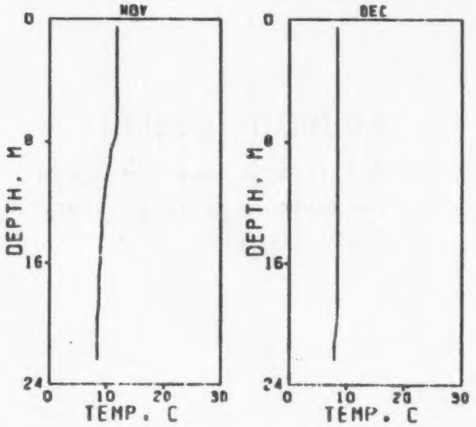




ATLANTA, GEORGIA

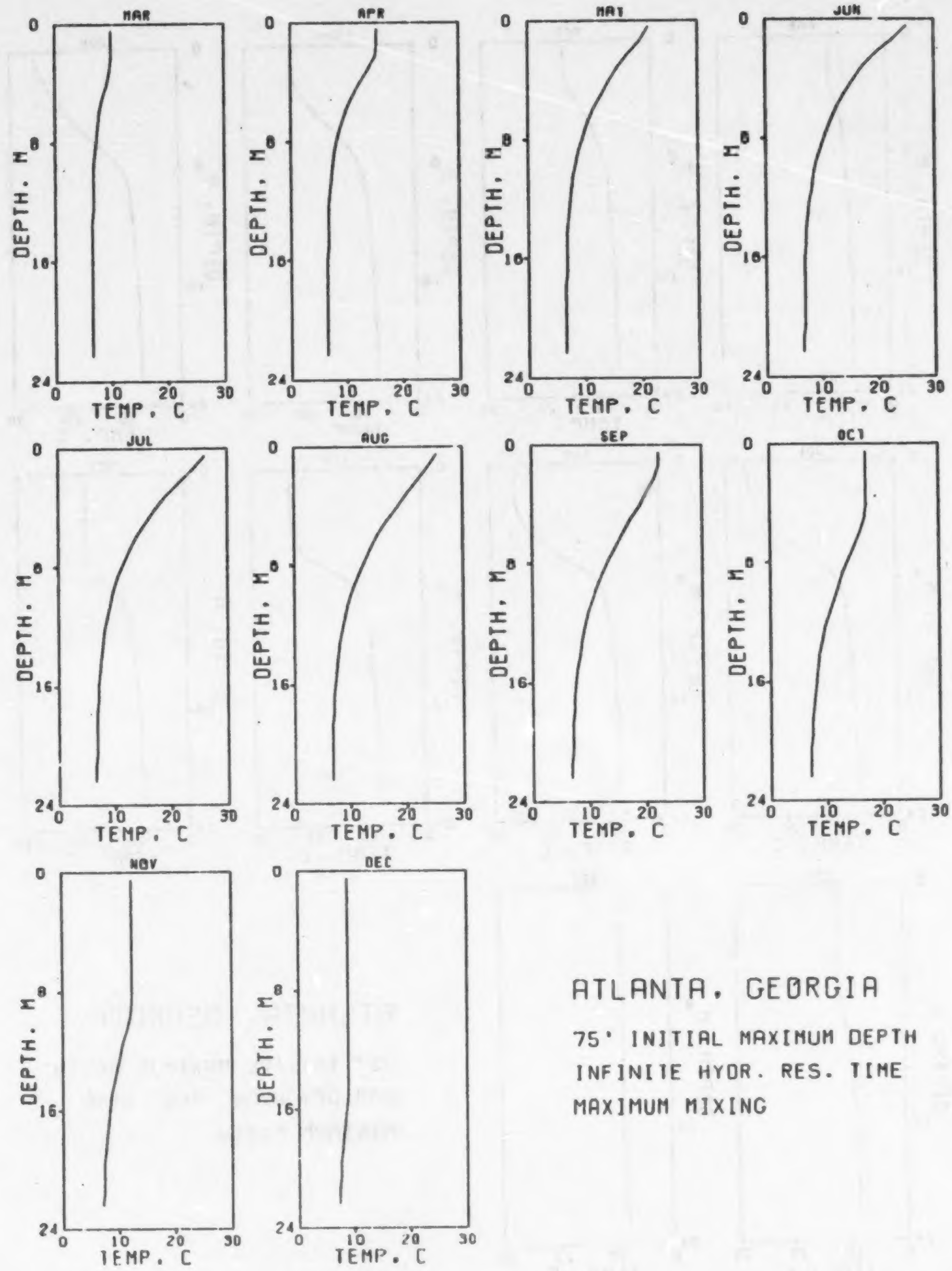
75' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING





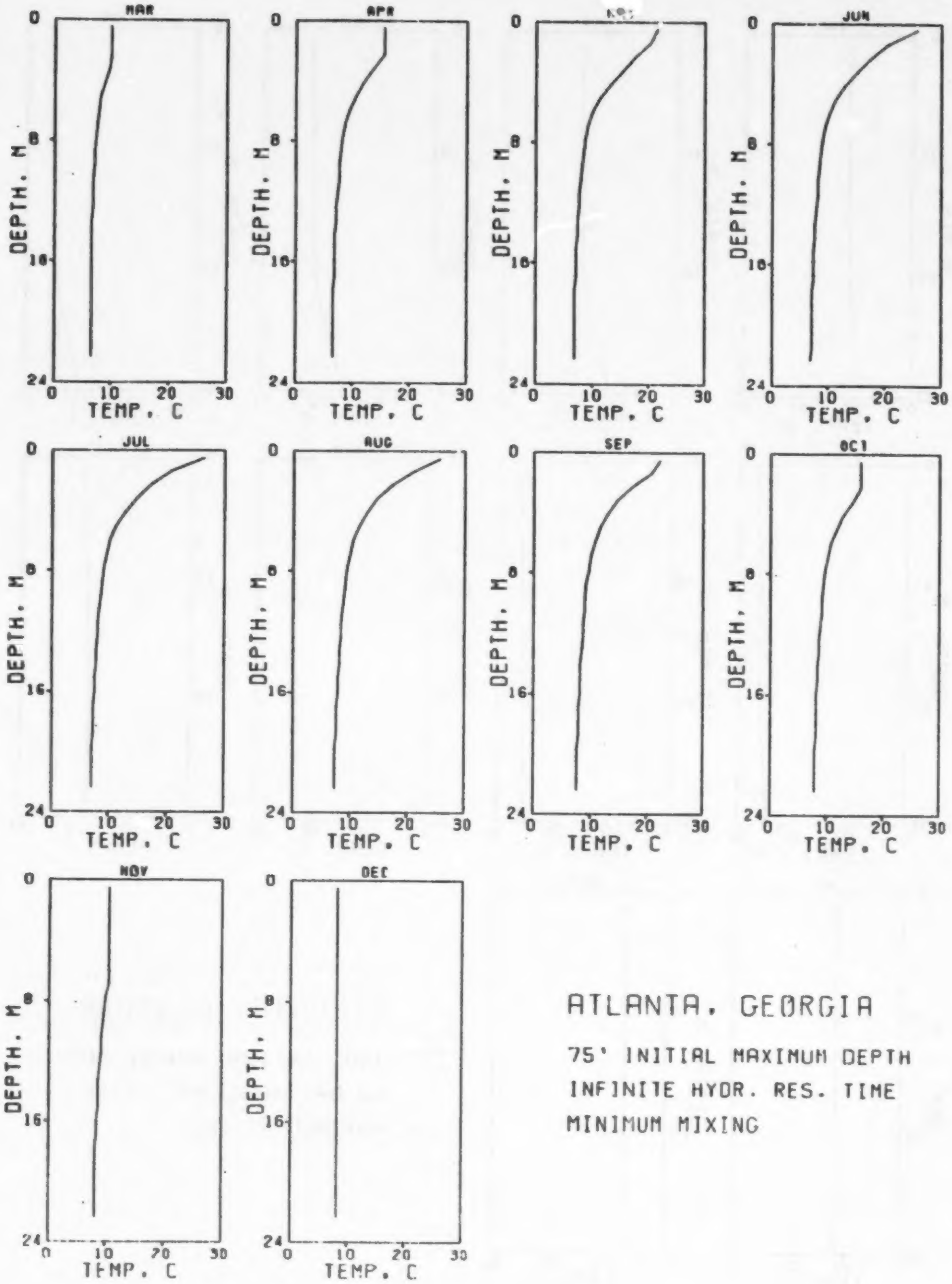
ATLANTA, GEORGIA

75' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



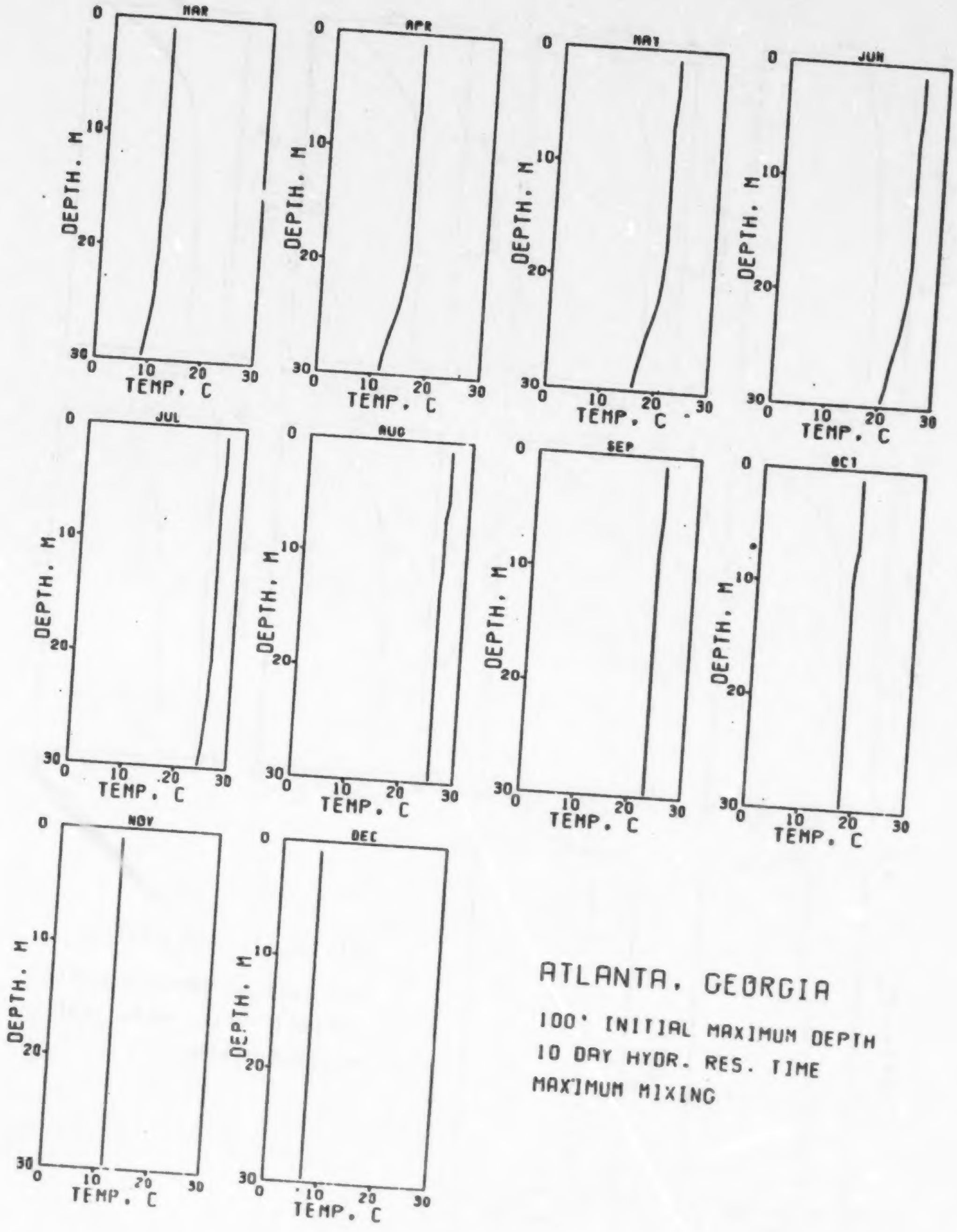
ATLANTA, GEORGIA

75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



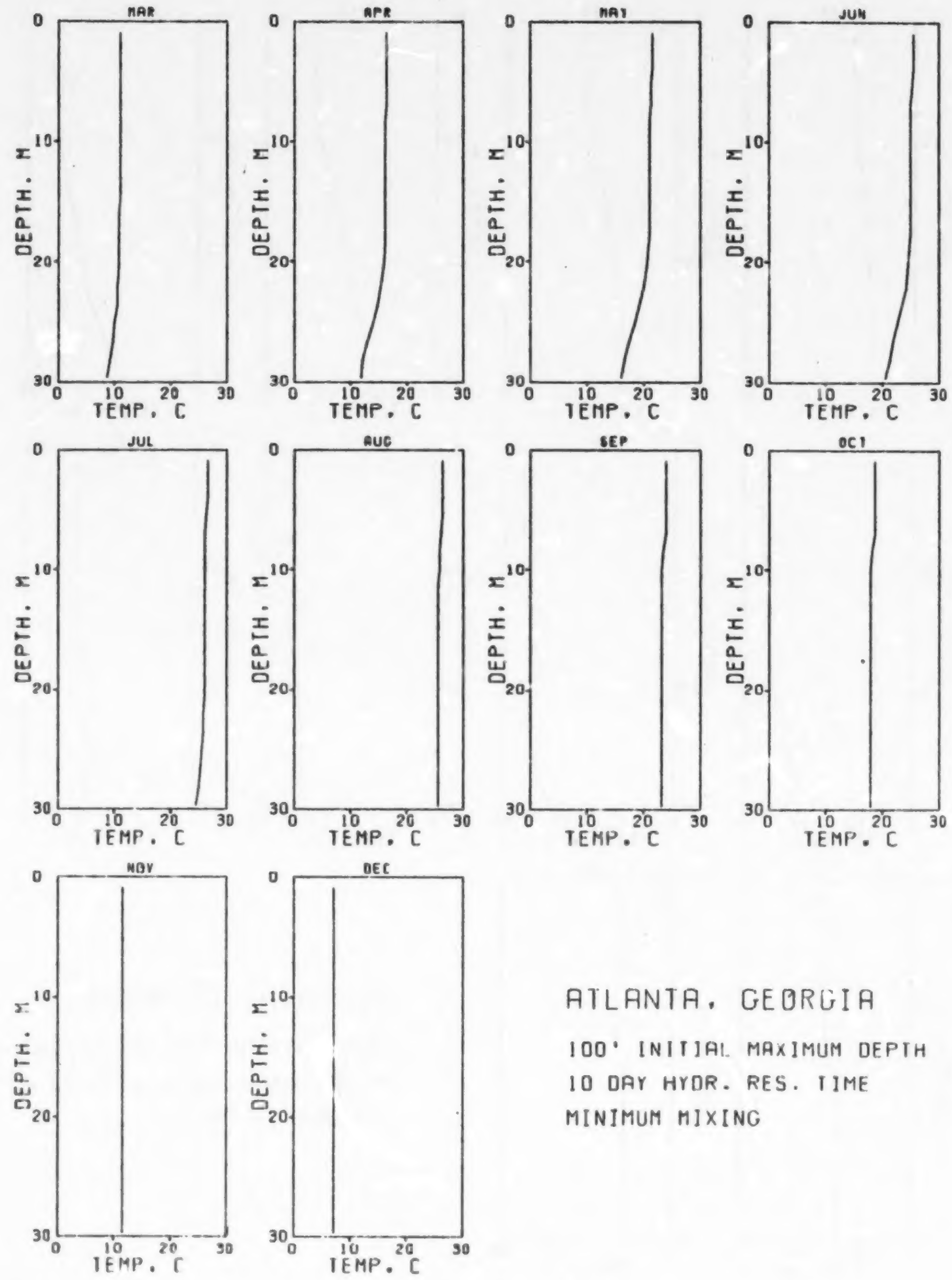
ATLANTA, GEORGIA

75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



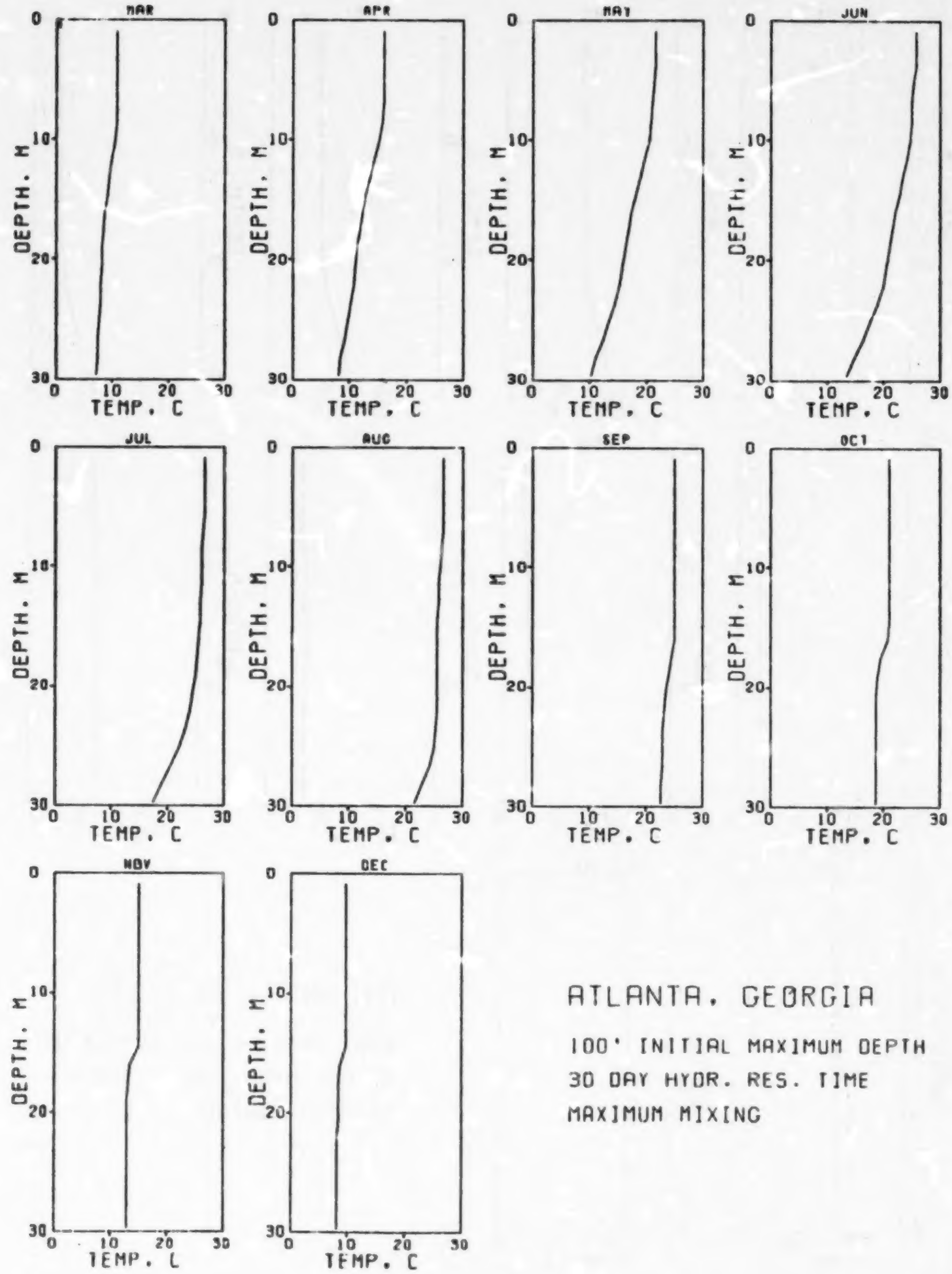
ATLANTA, GEORGIA

100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



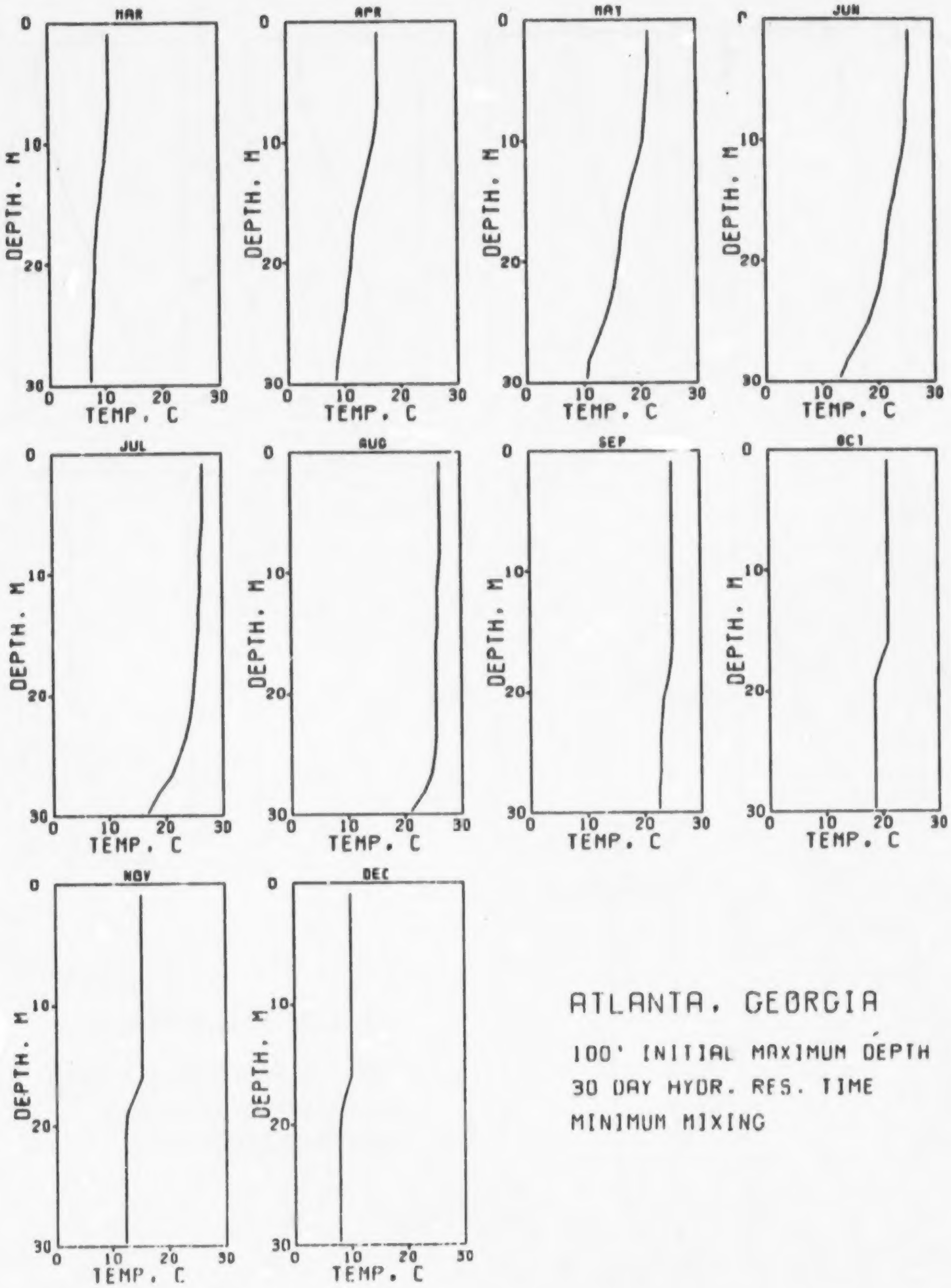
ATLANTA, GEORGIA

100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



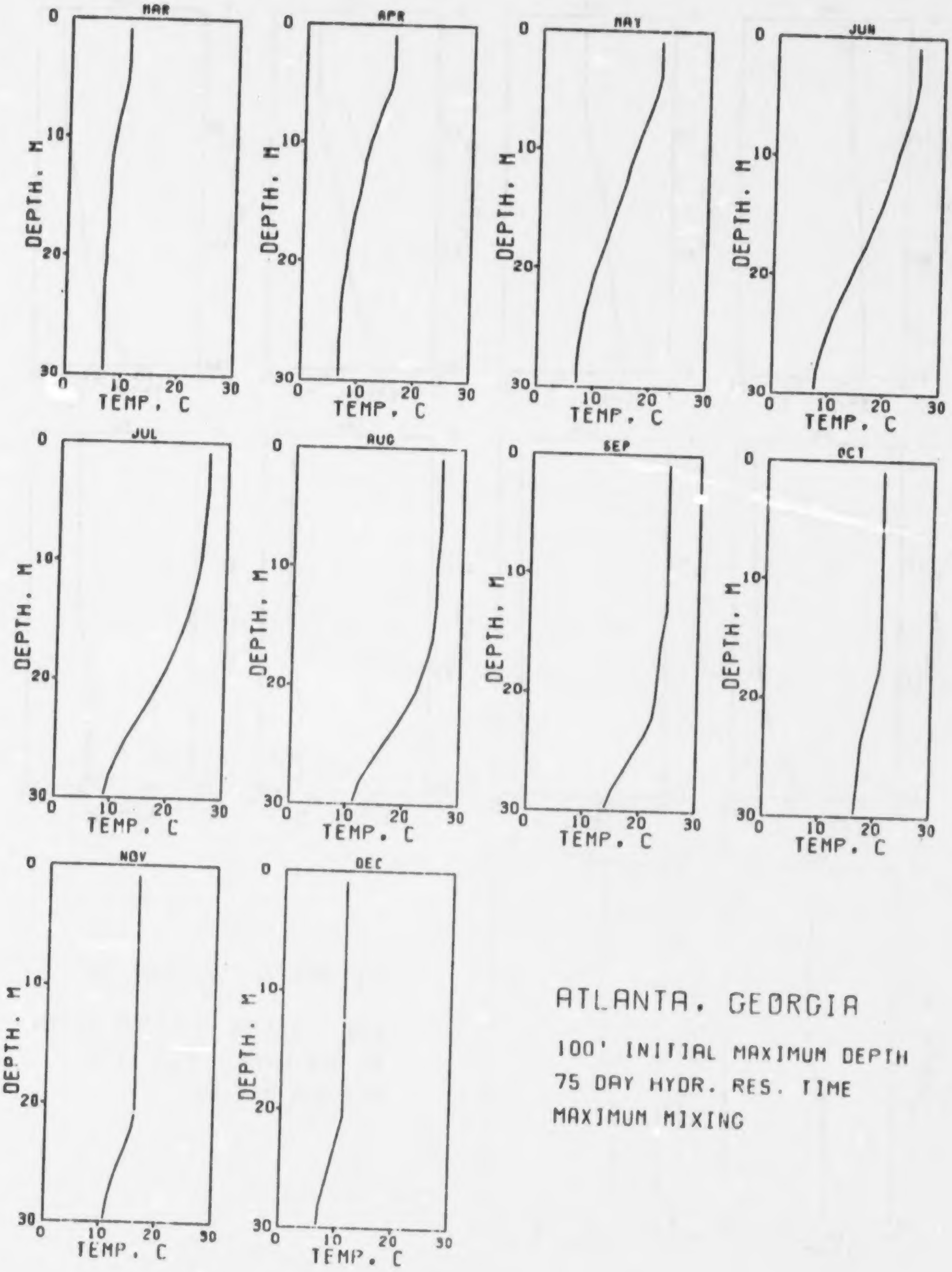
ATLANTA, GEORGIA

100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



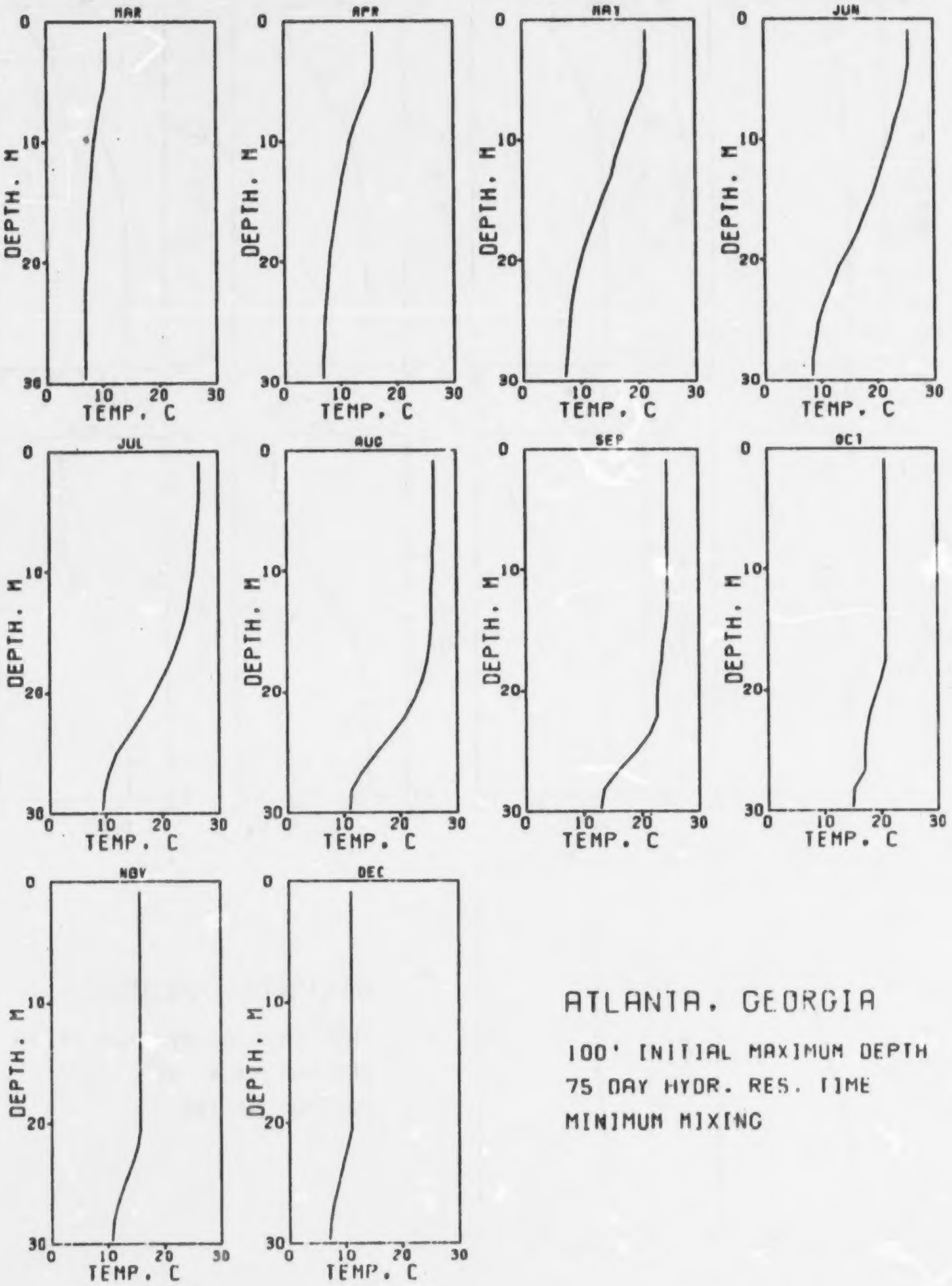
ATLANTA, GEORGIA

100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



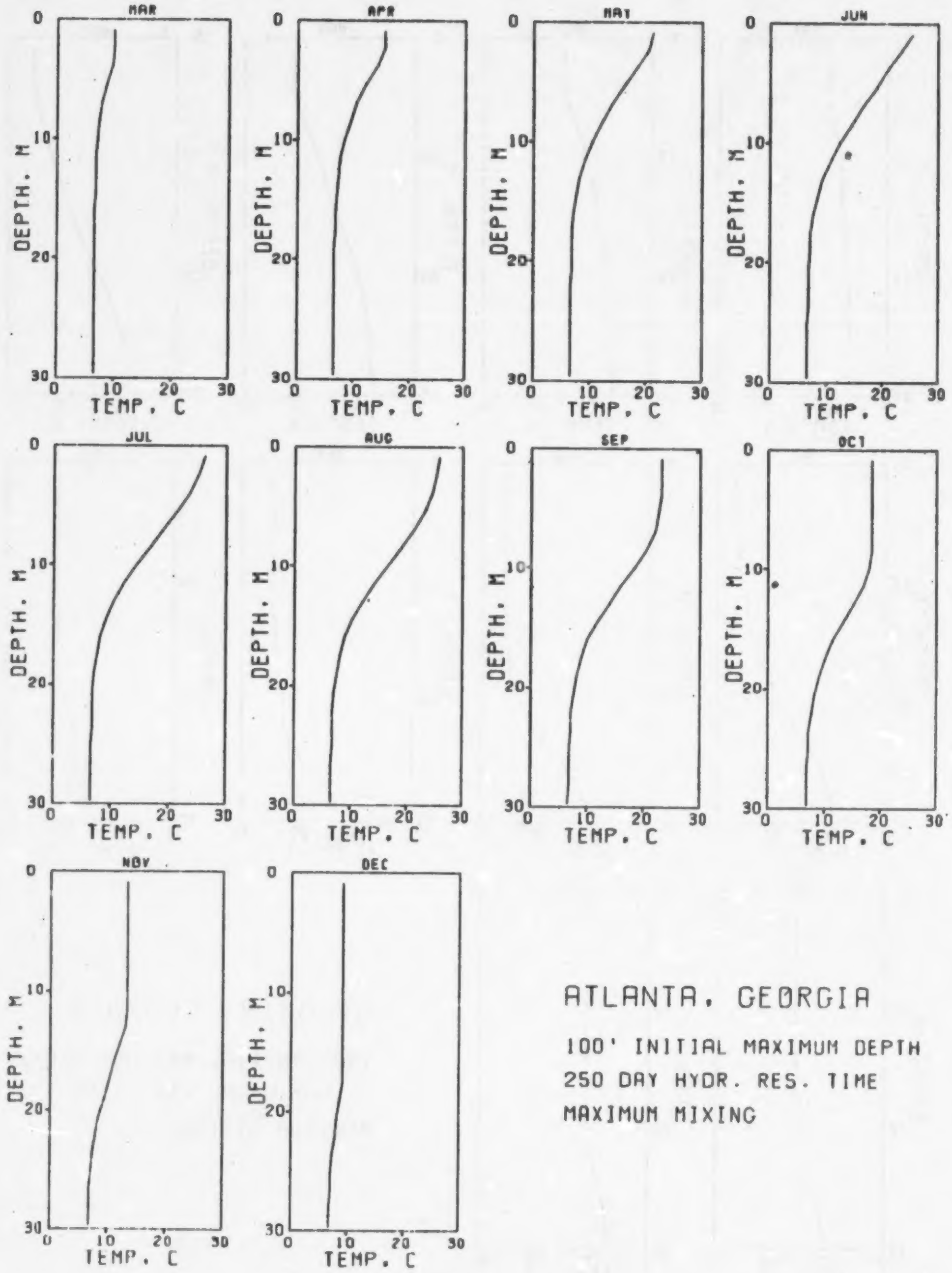
ATLANTA, GEORGIA

100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



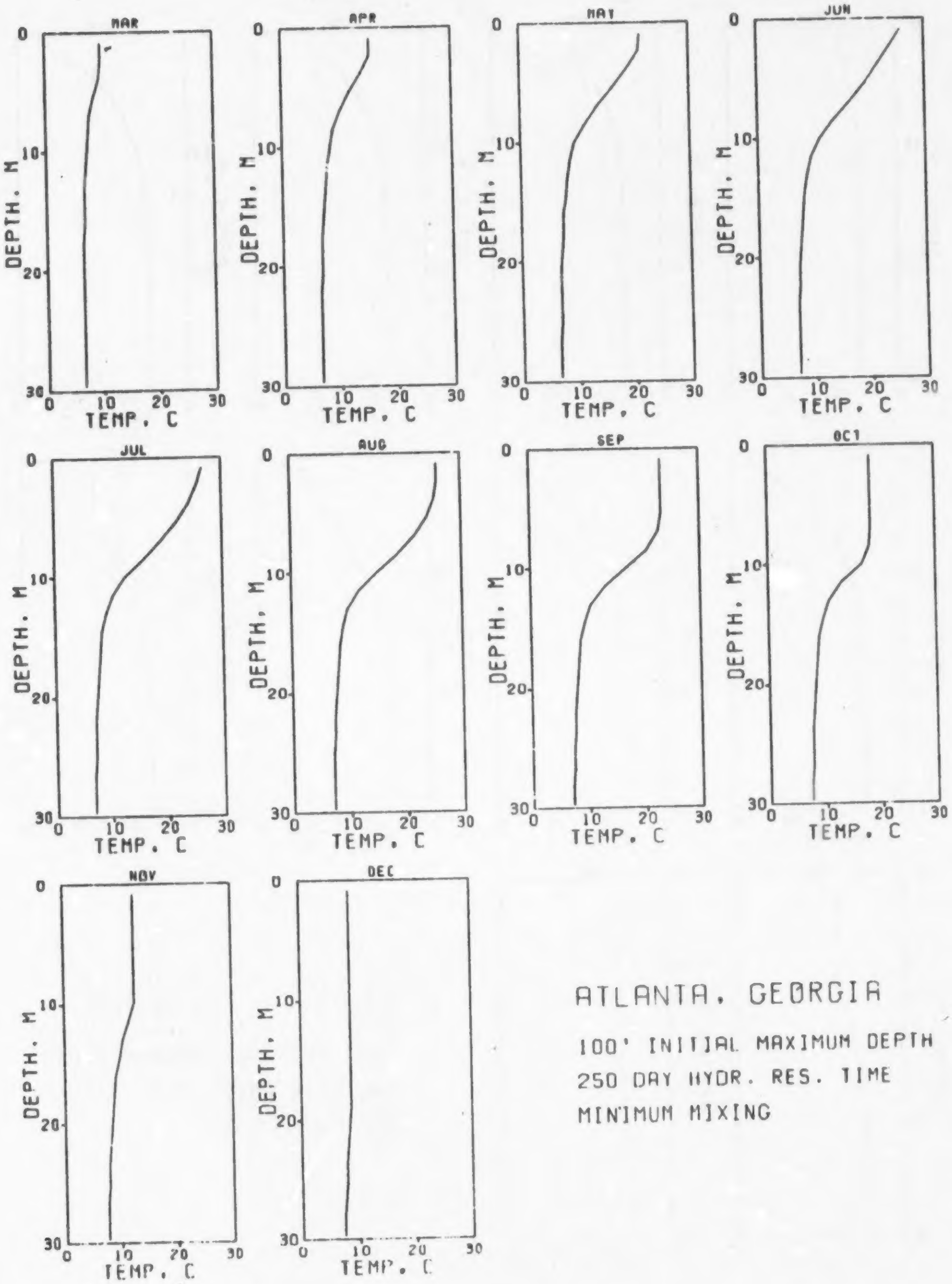
ATLANTA, GEORGIA

100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



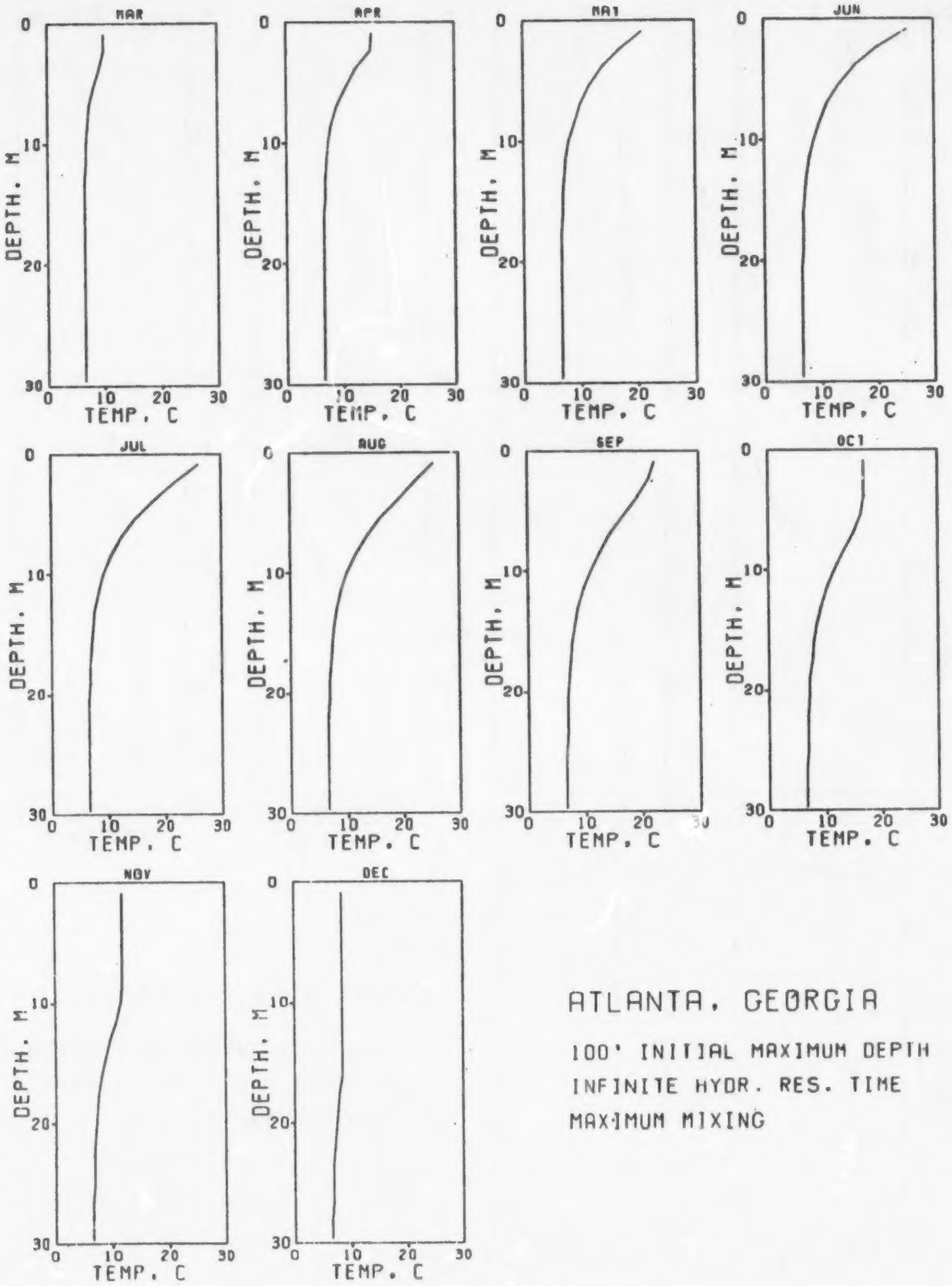
ATLANTA, GEORGIA

100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



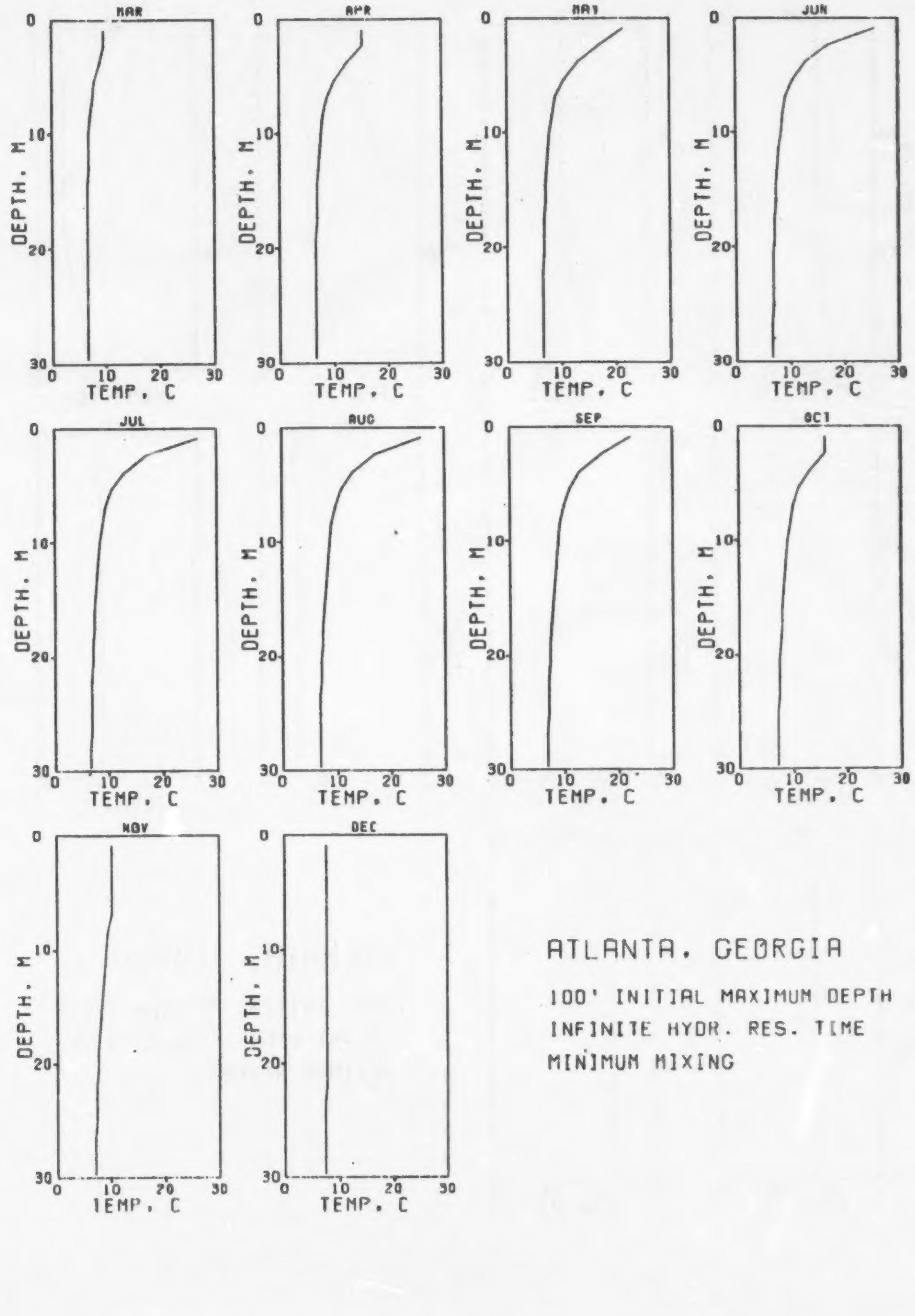
ATLANTA, GEORGIA

100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



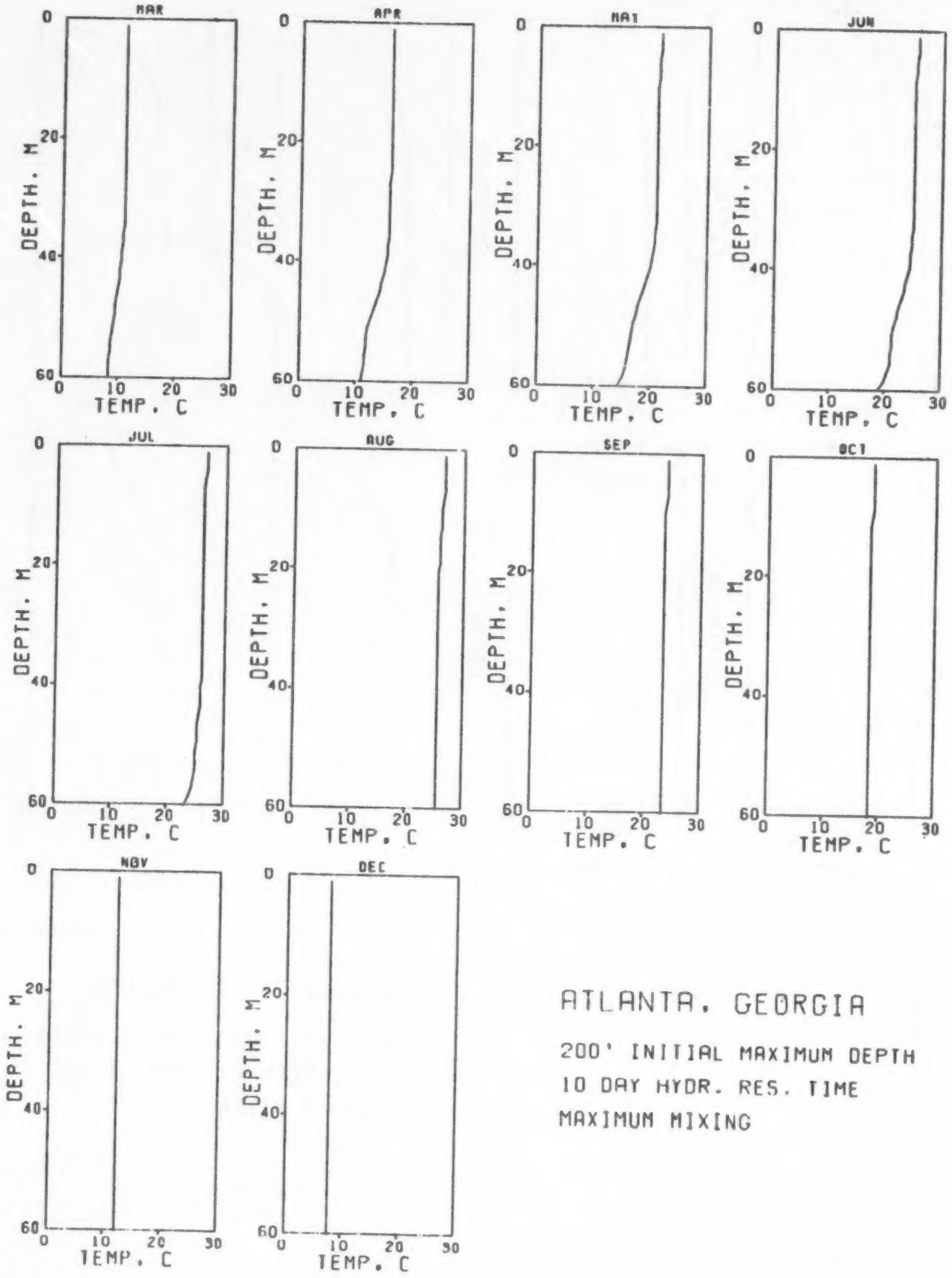
ATLANTA, GEORGIA

100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



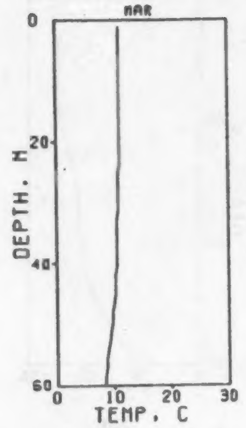
ATLANTA, GEORGIA

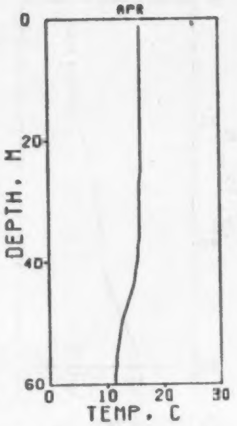
100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING

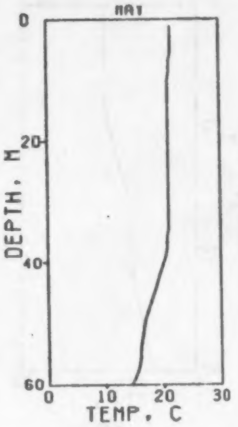


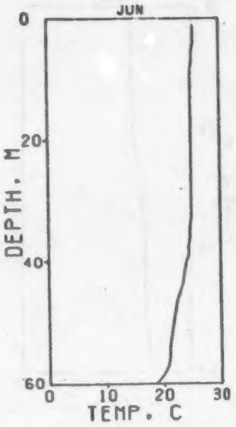
ATLANTA, GEORGIA

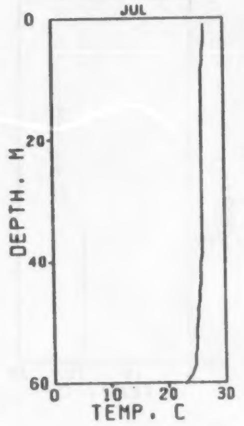
200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING

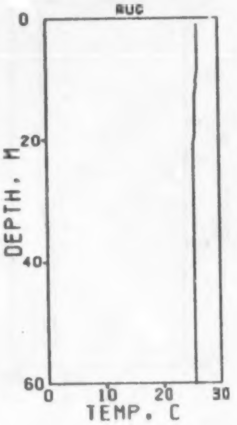


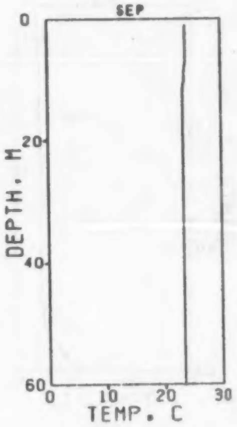


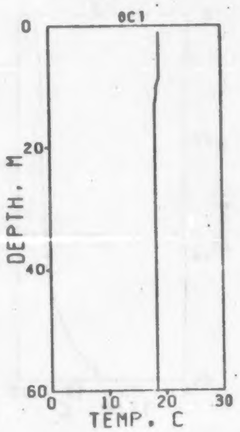


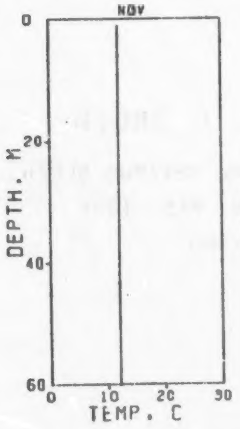


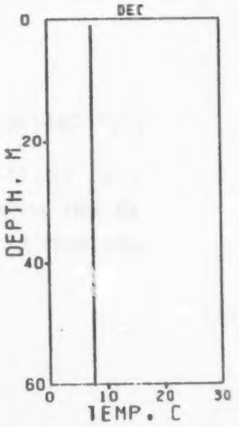






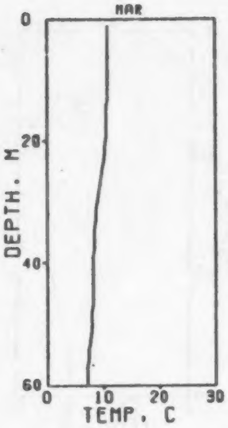


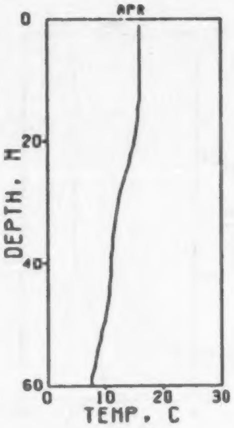


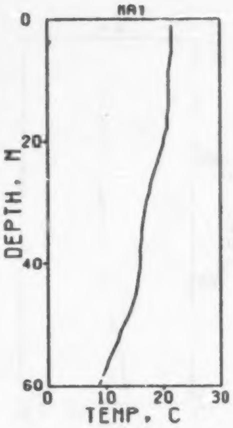


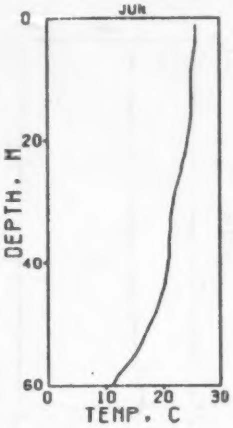
ATLANTA, GEORGIA

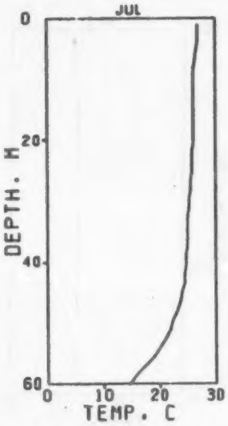
200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING

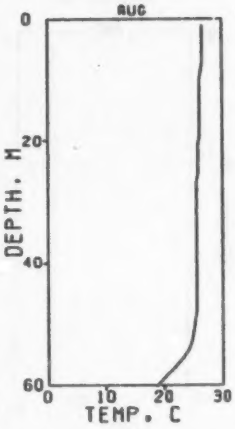


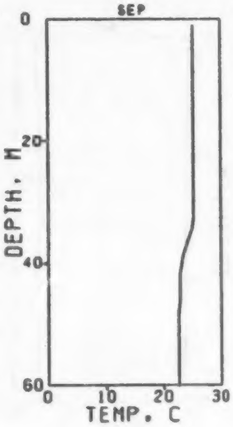


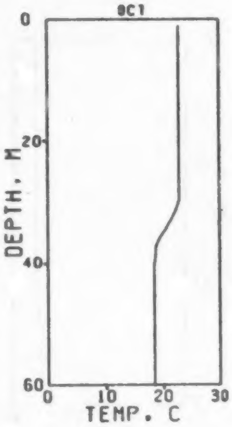


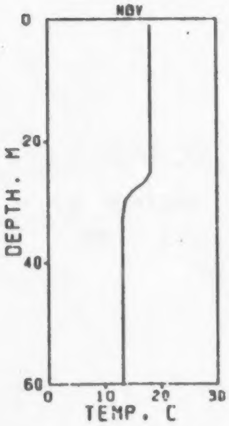


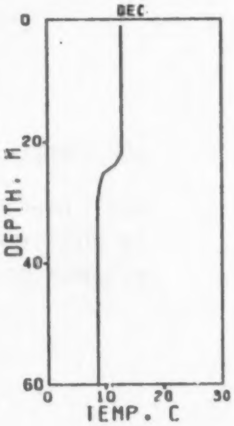






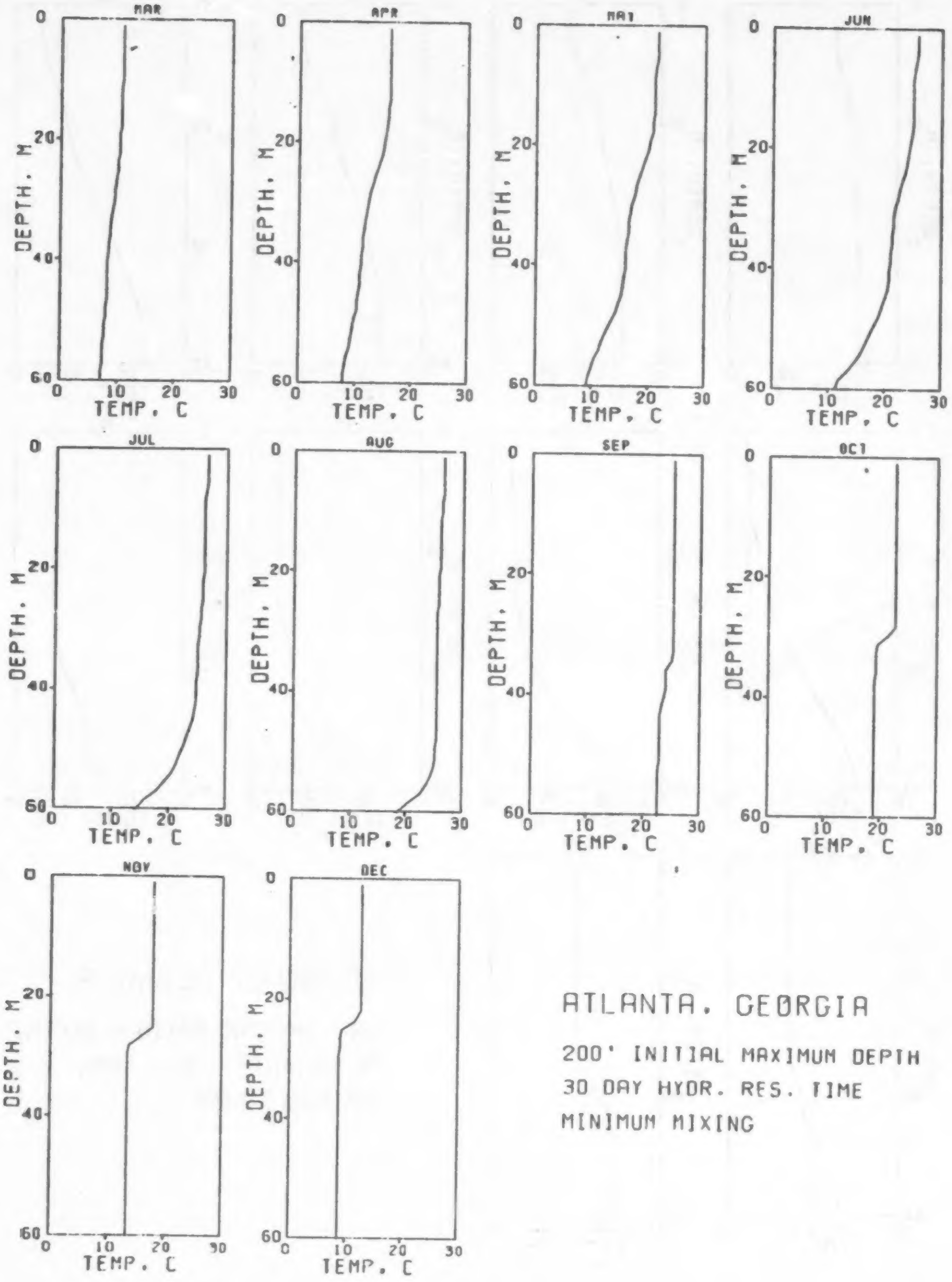






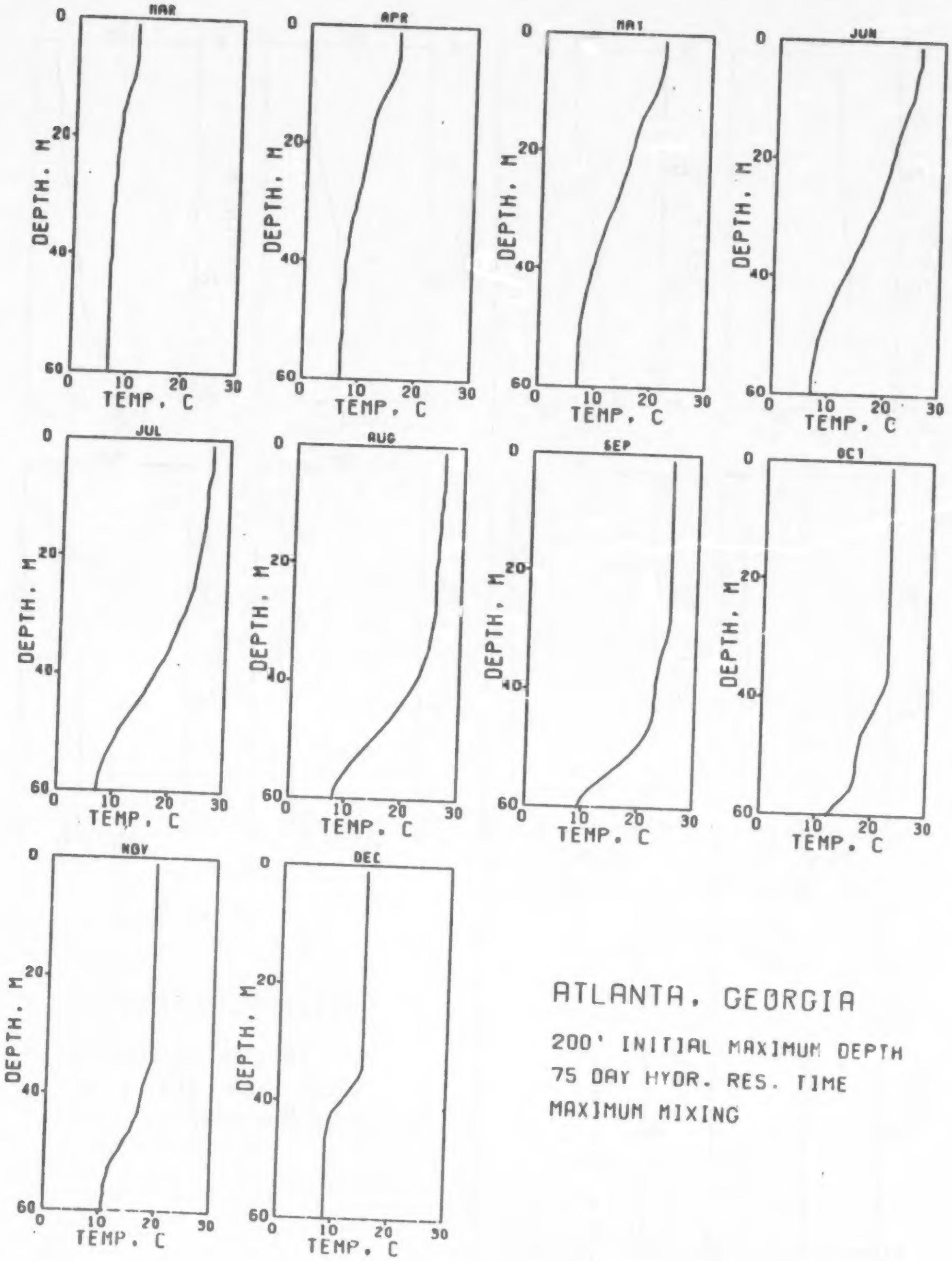
ATLANTA, GEORGIA

200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



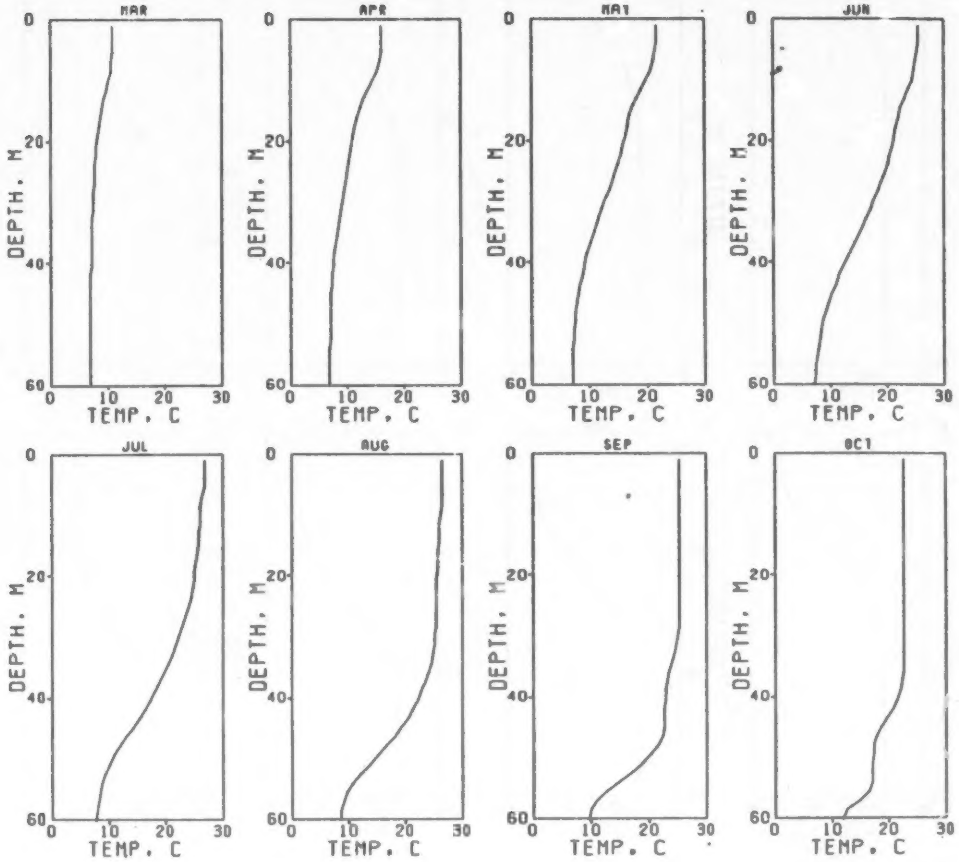
ATLANTA, GEORGIA

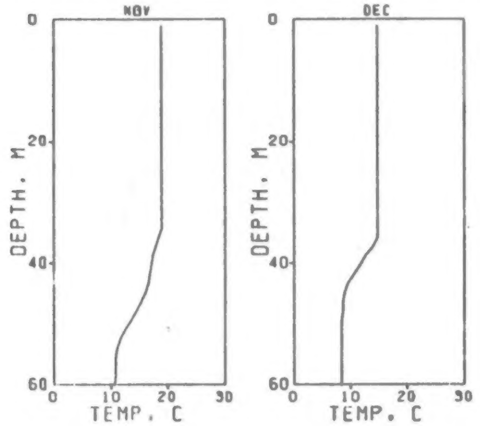
200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



ATLANTA, GEORGIA

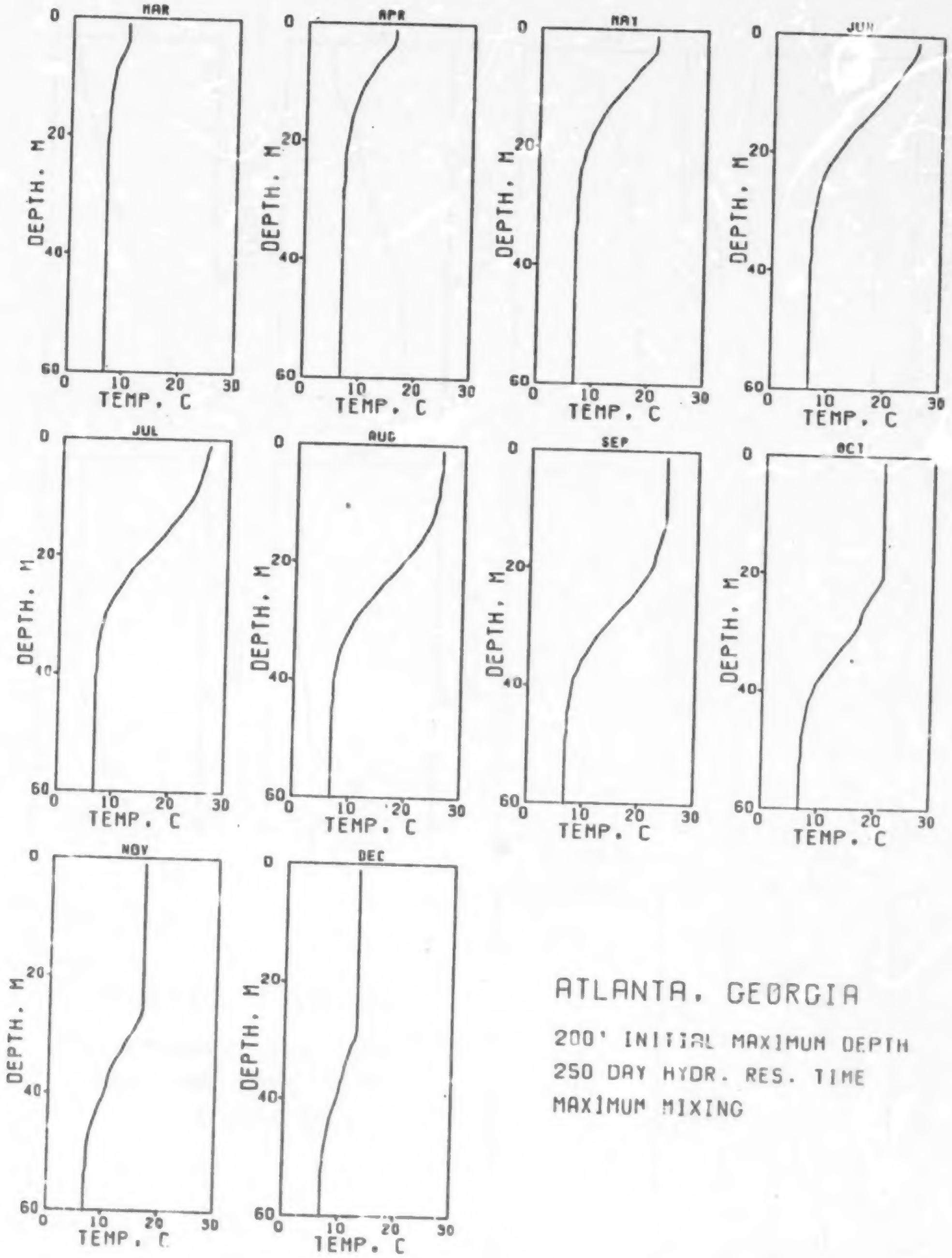
200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING





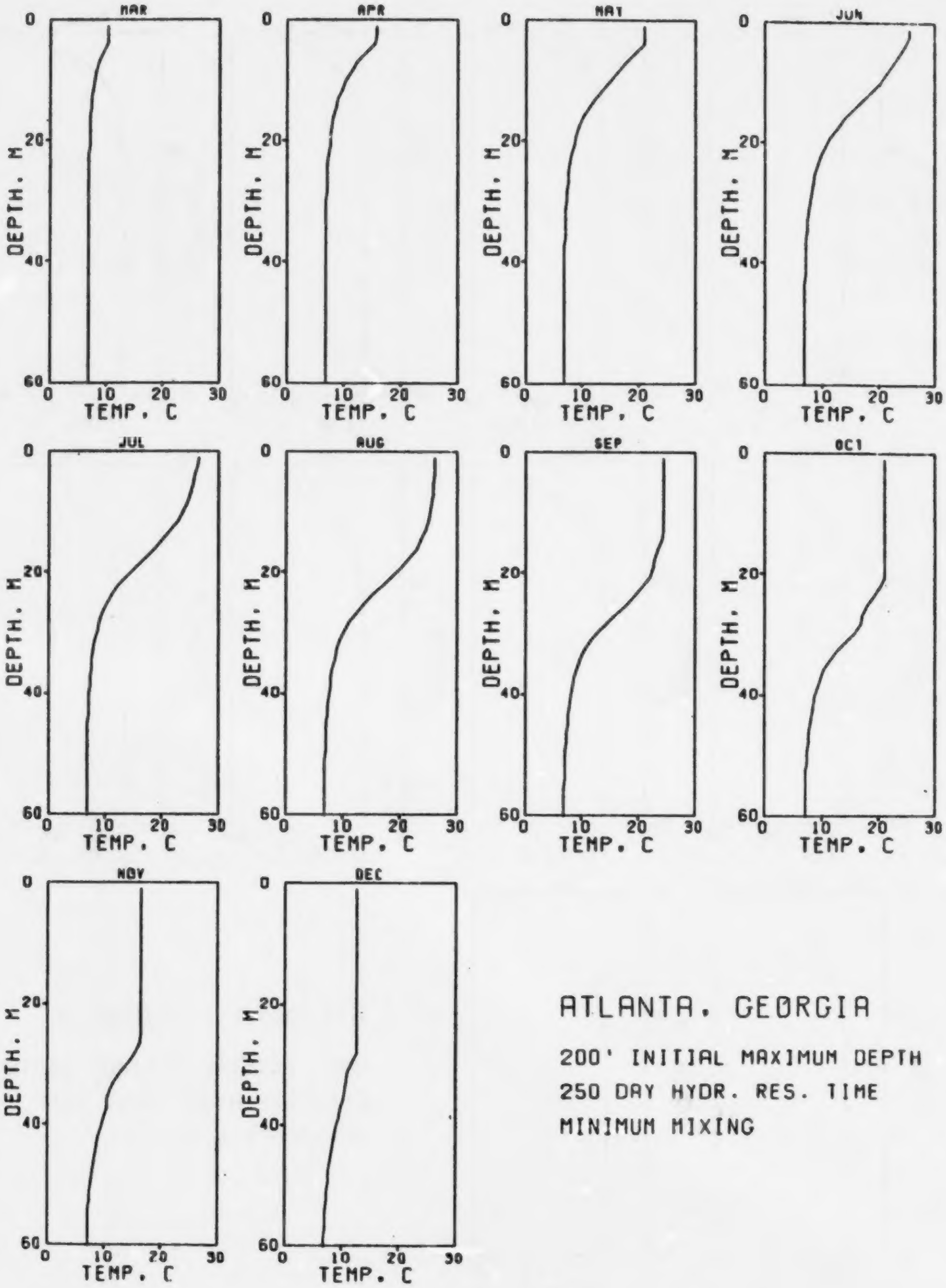
ATLANTA, GEORGIA

200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



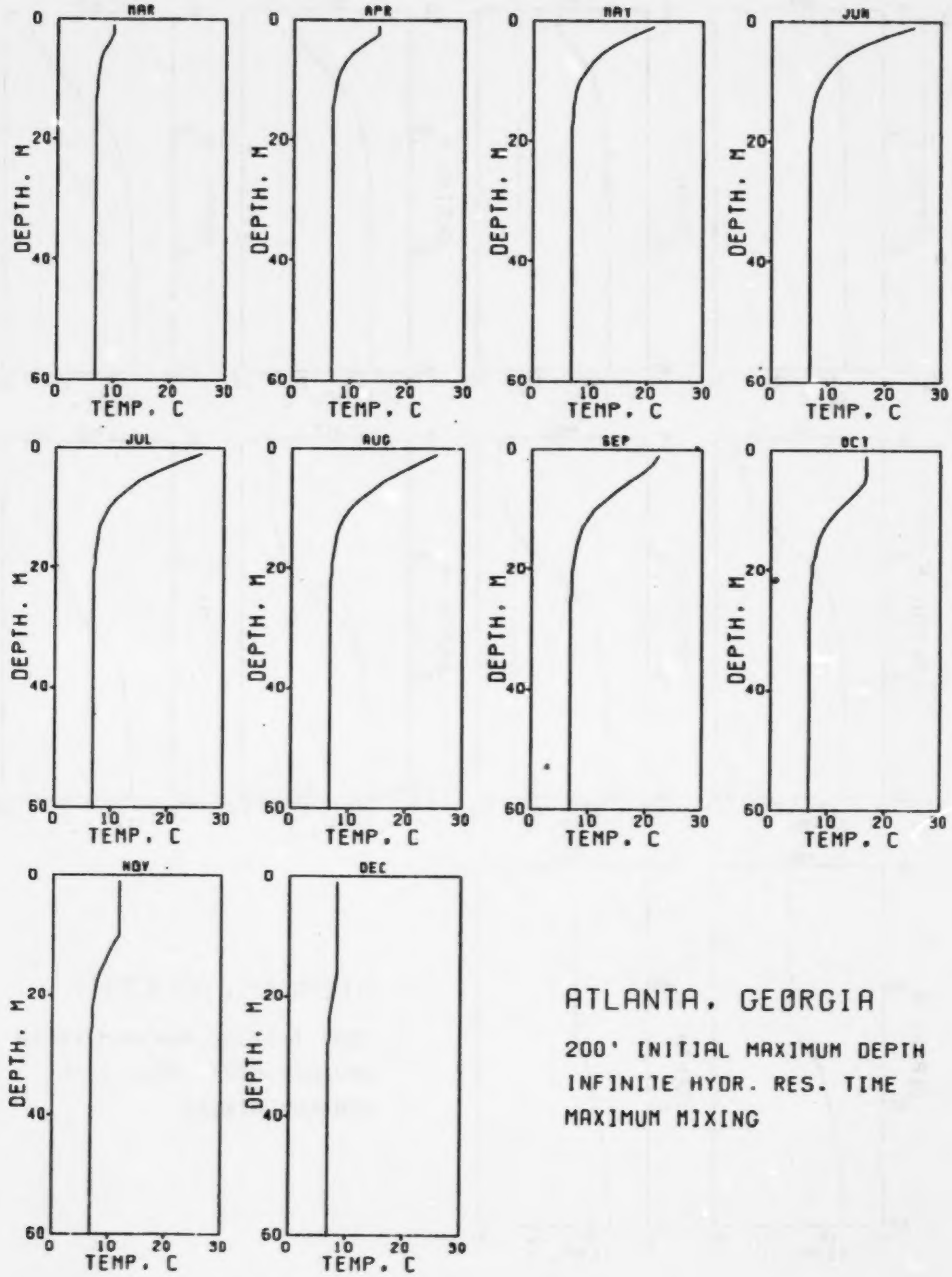
ATLANTA, GEORGIA

200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



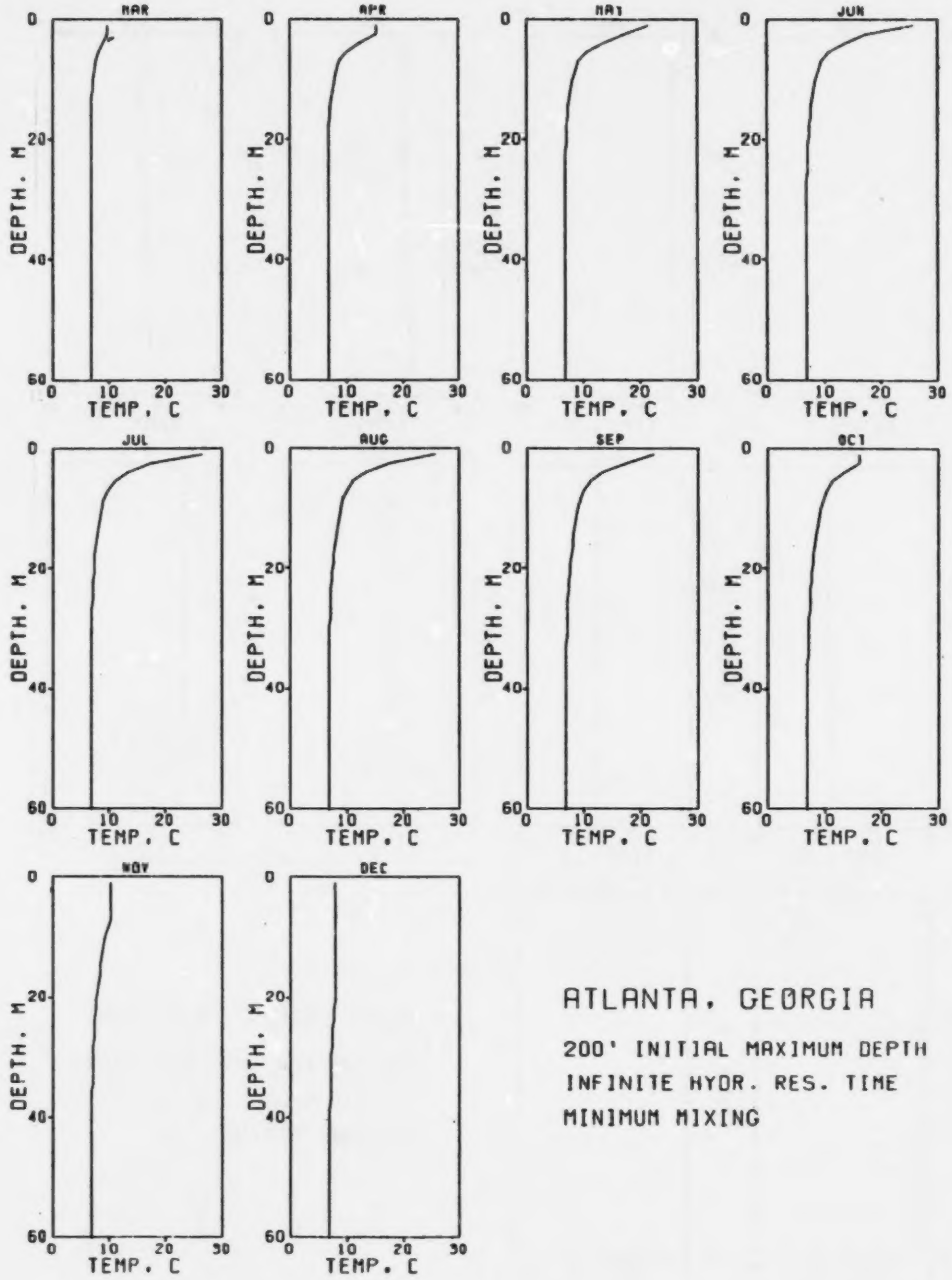
ATLANTA, GEORGIA

200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



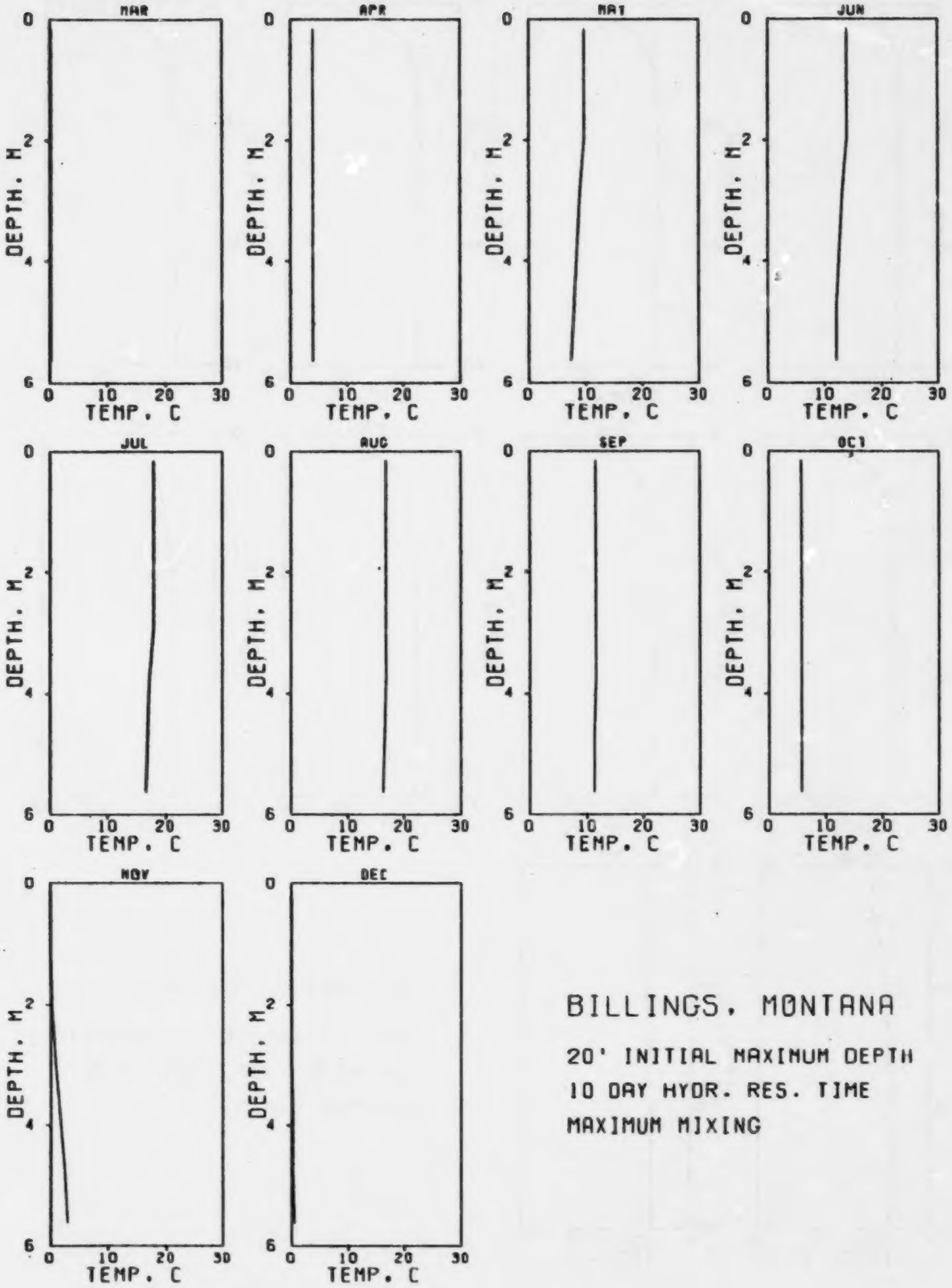
ATLANTA, GEORGIA

200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



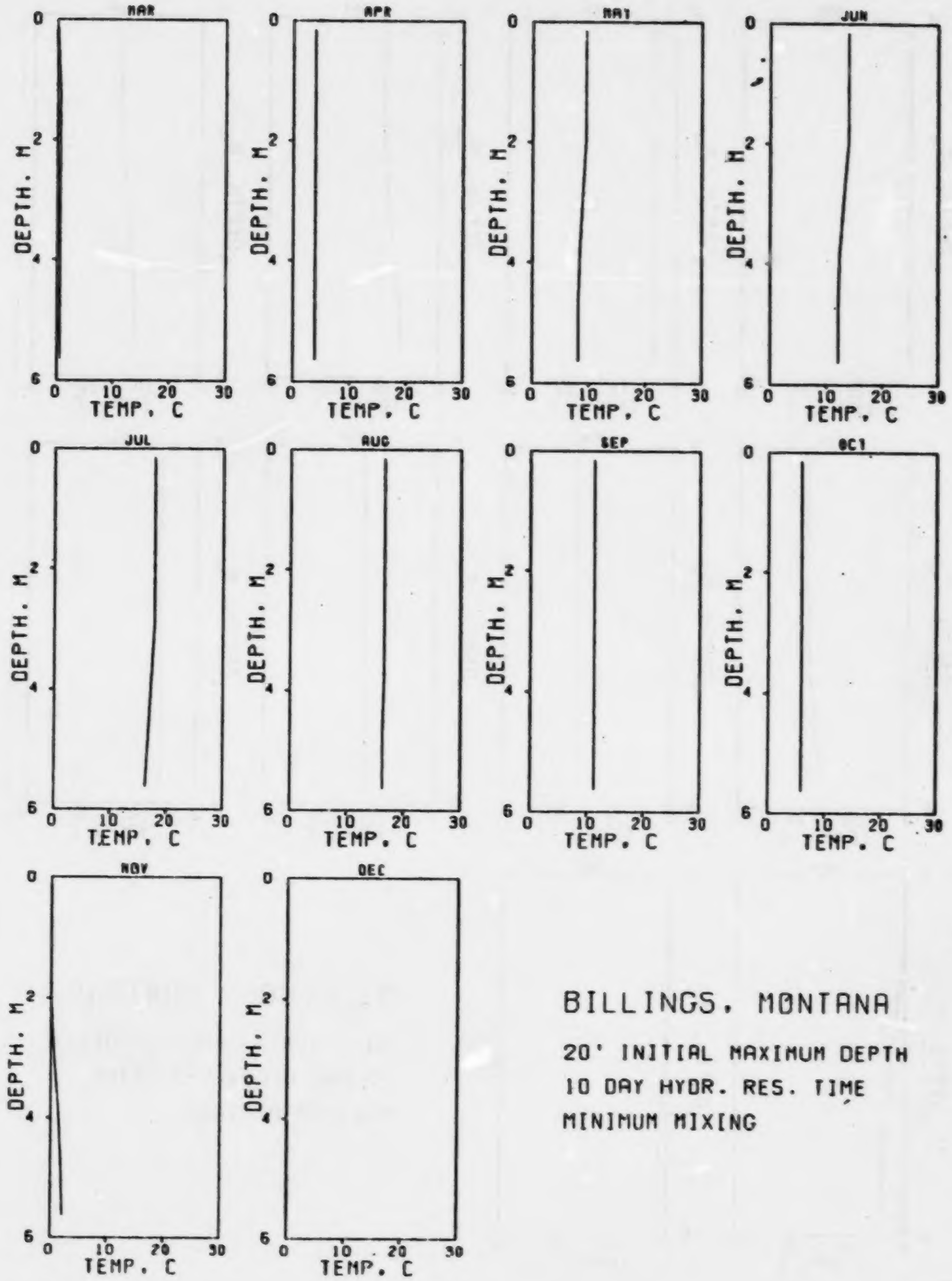
ATLANTA, GEORGIA

200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



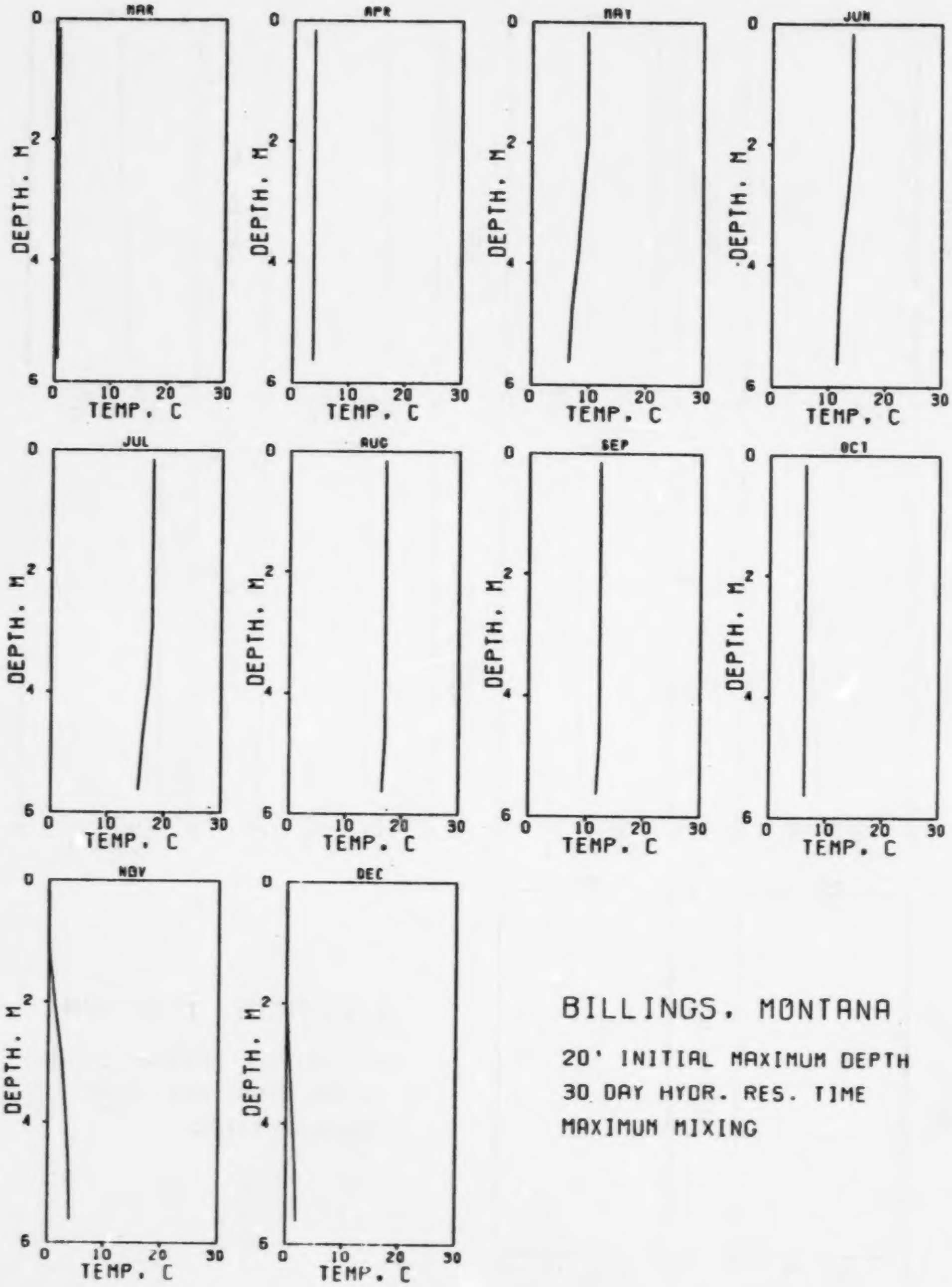
BILLINGS, MONTANA

20' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



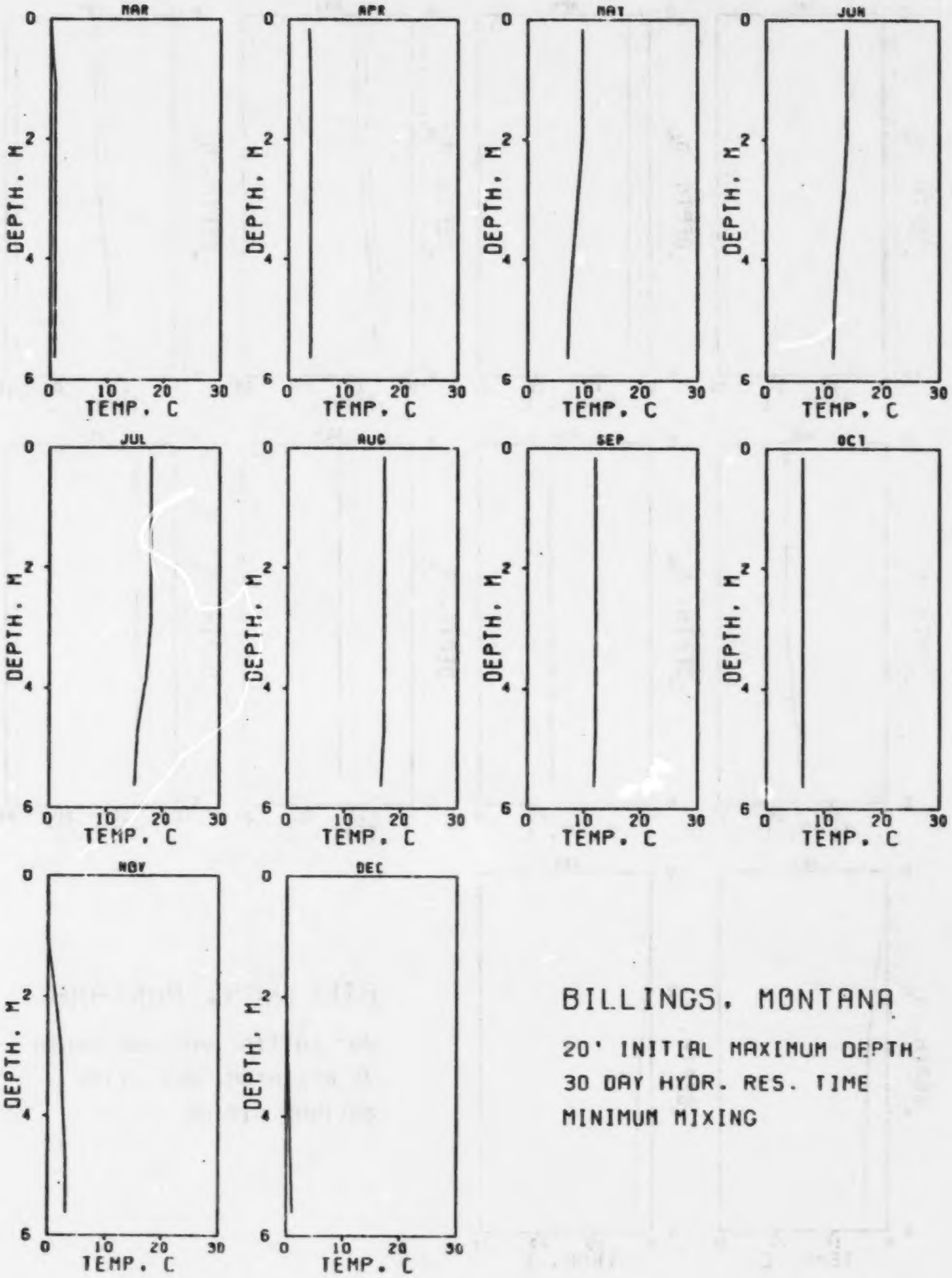
BILLINGS, MONTANA

20' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



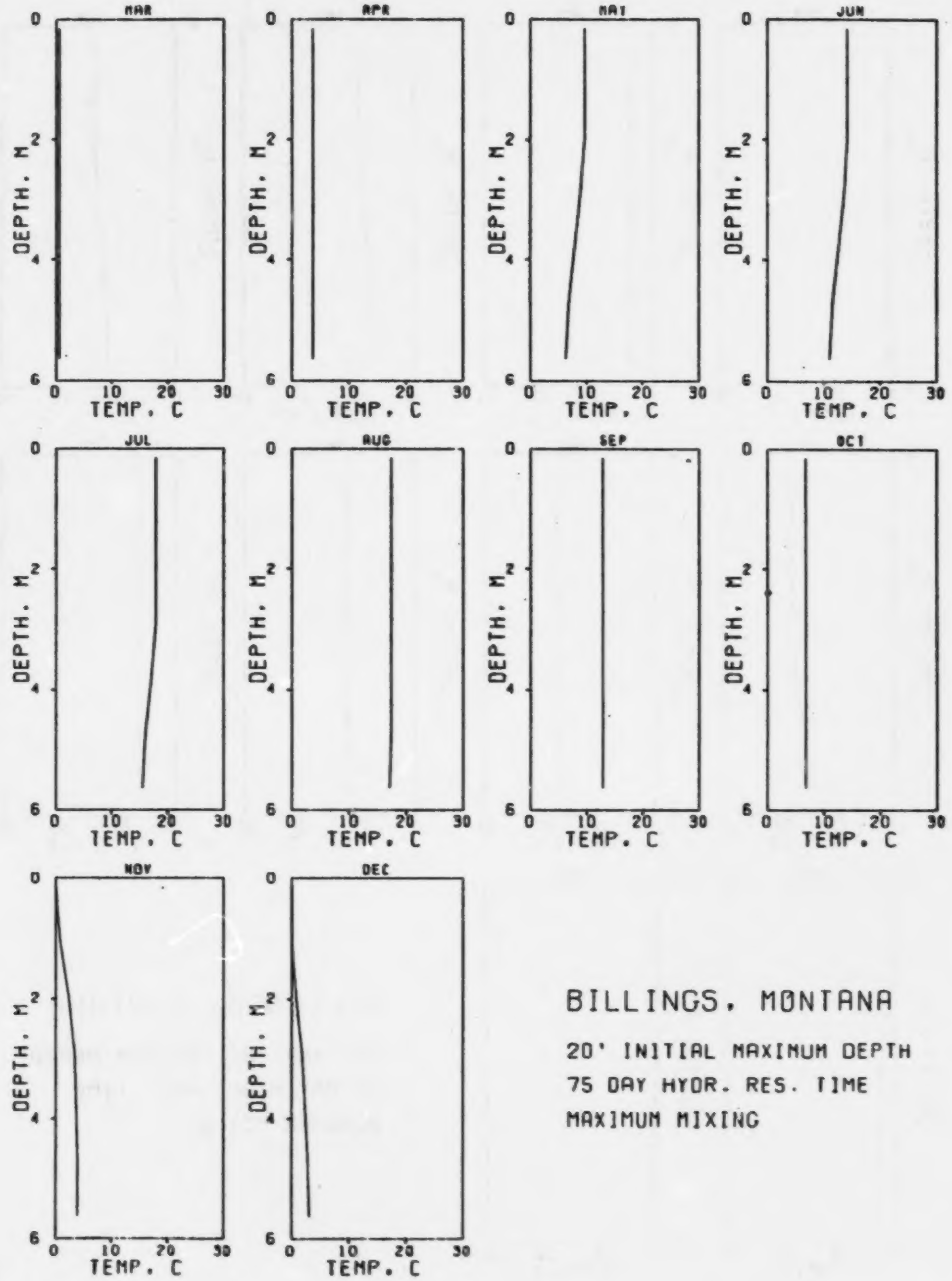
BILLINGS, MONTANA

20' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



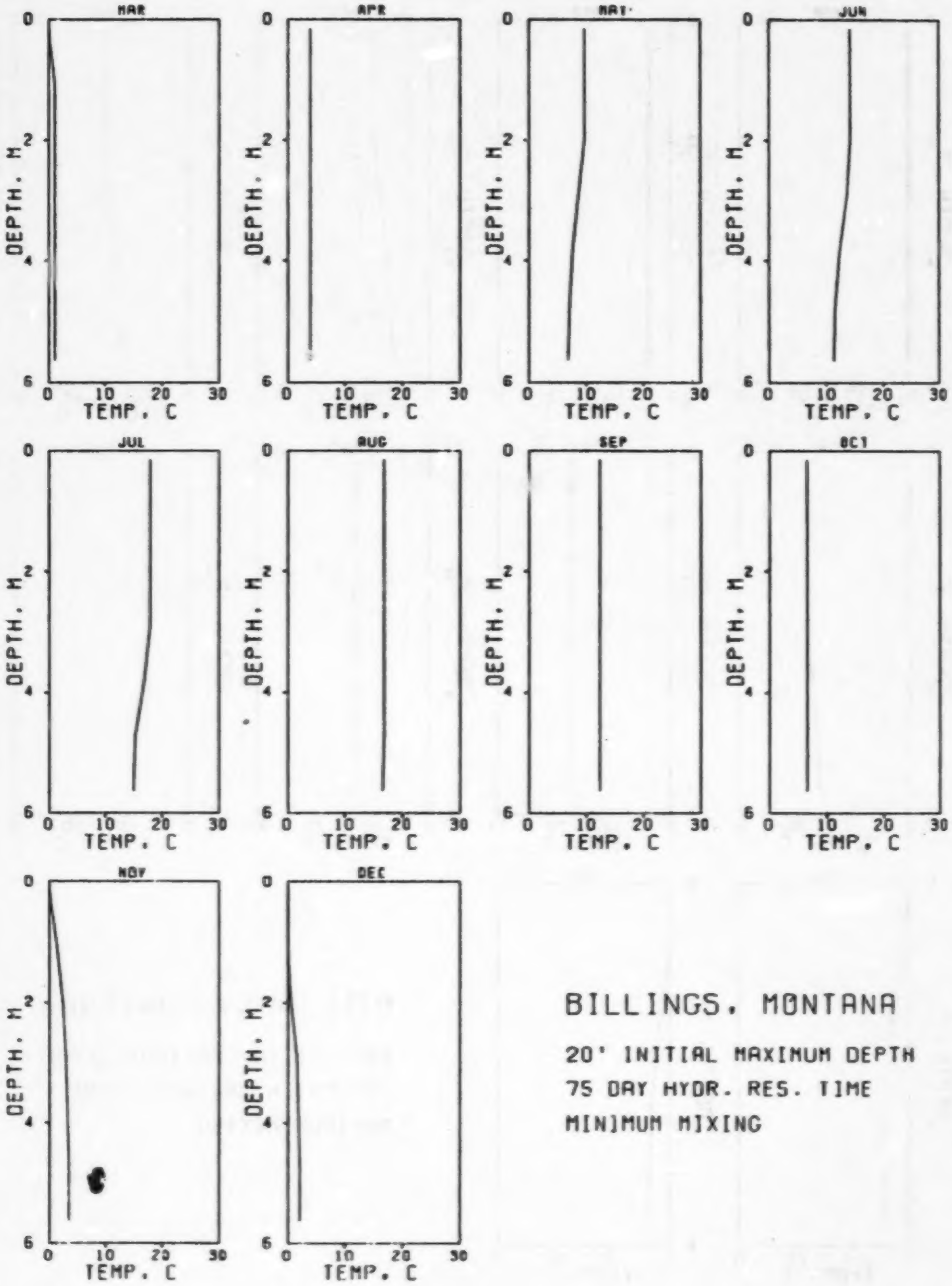
BILLINGS, MONTANA

20' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



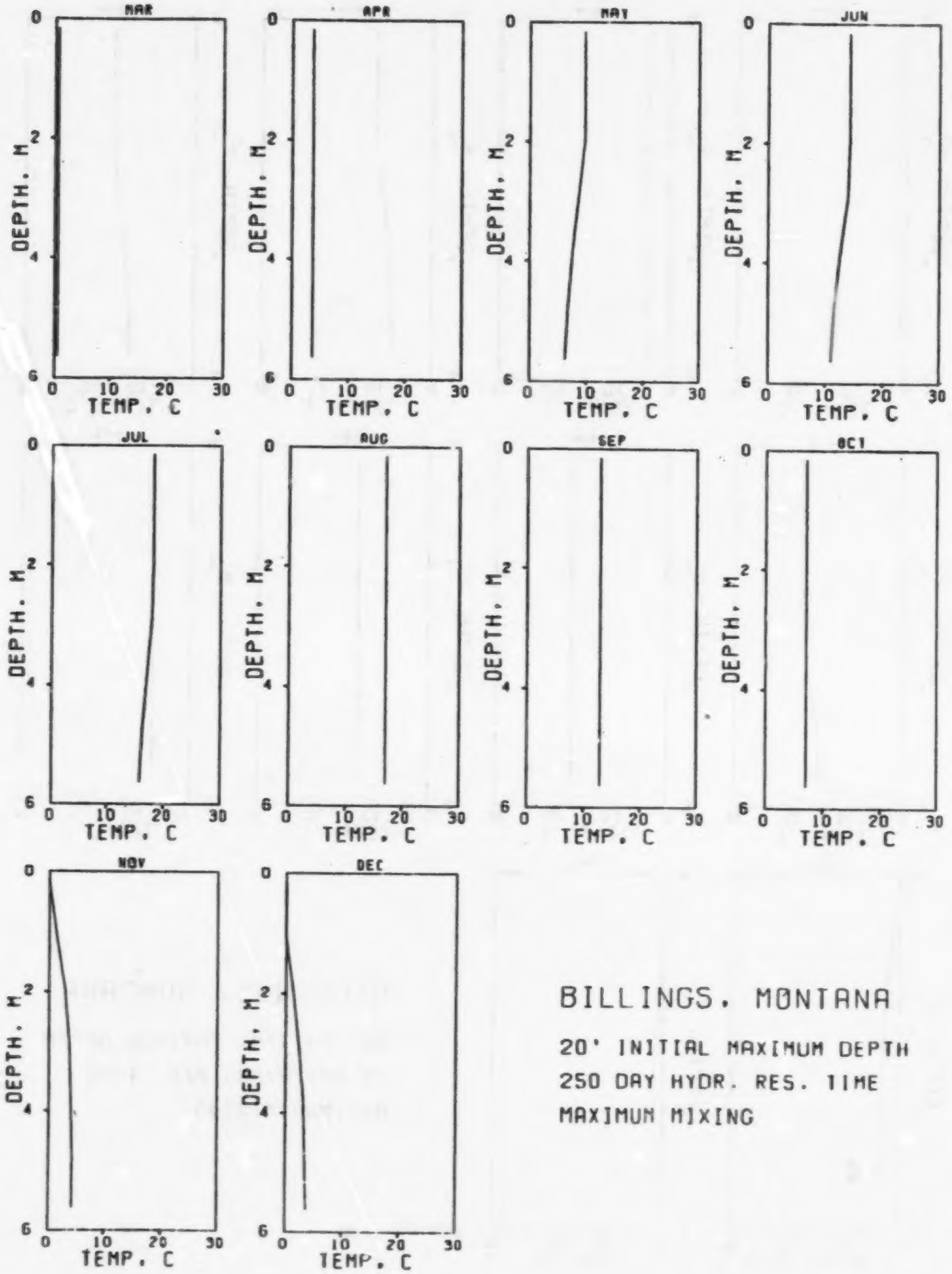
BILLINGS, MONTANA

20' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



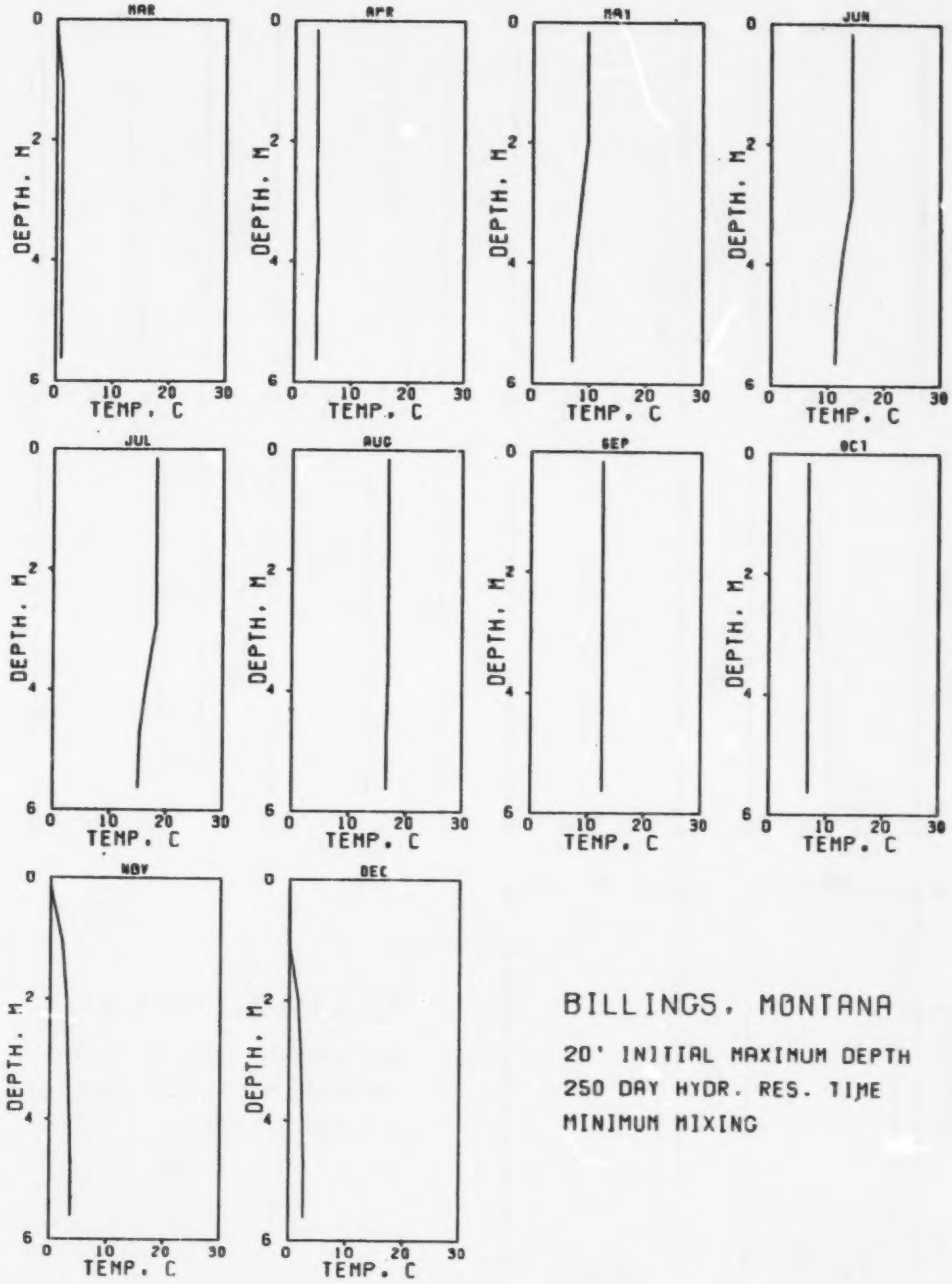
BILLINGS, MONTANA

20' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



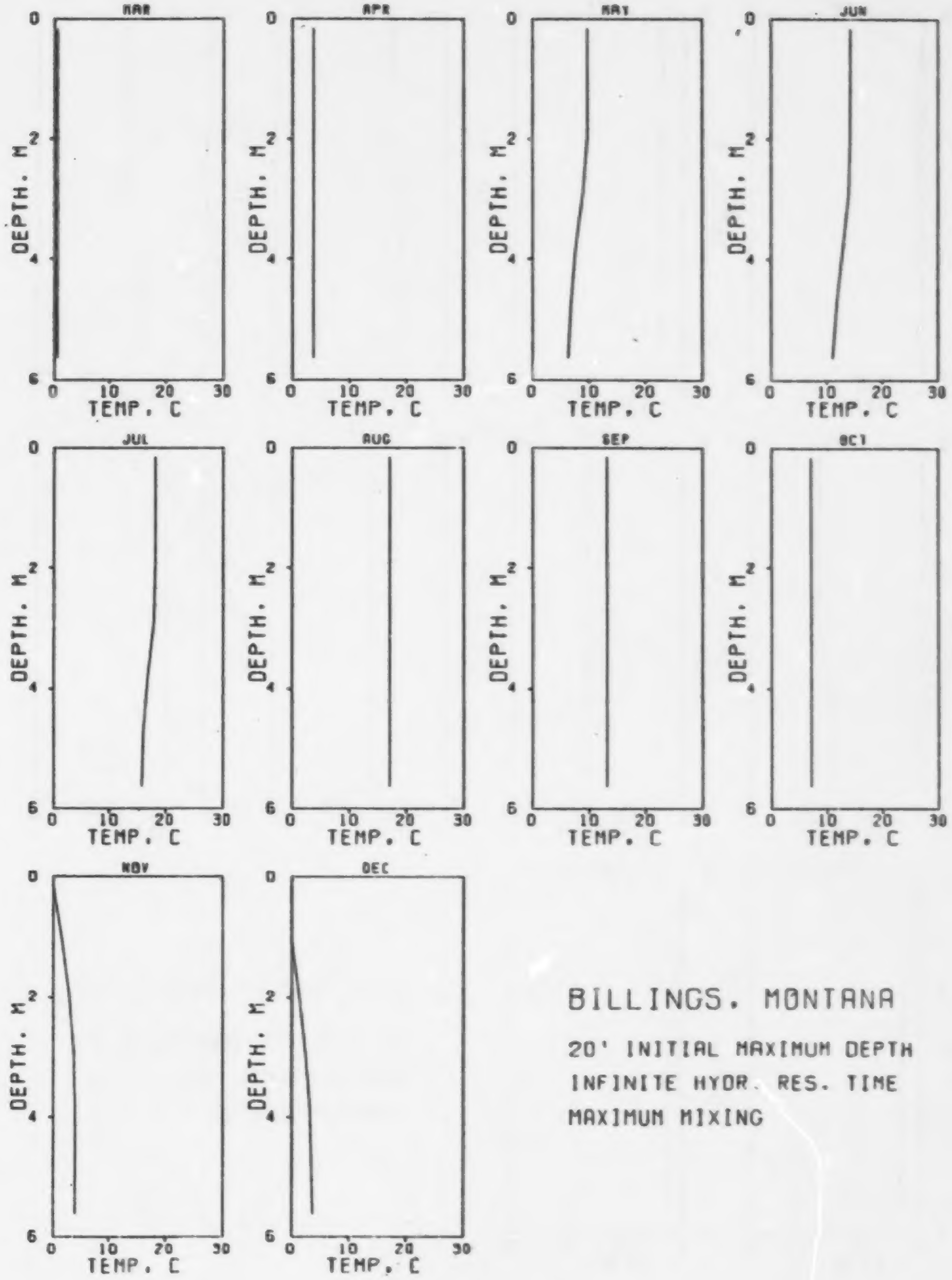
BILLINGS, MONTANA

20' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



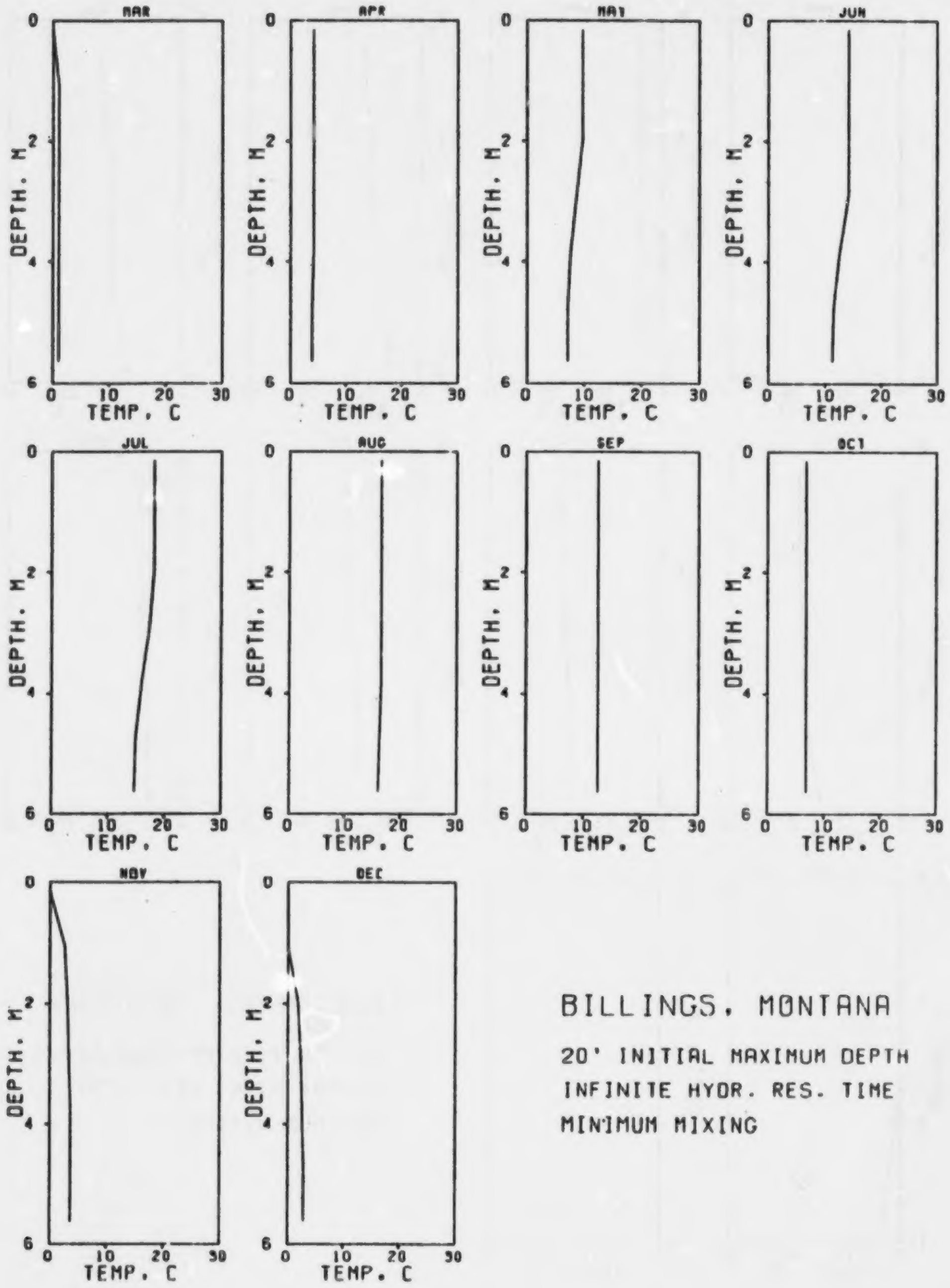
BILLINGS, MONTANA

20' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



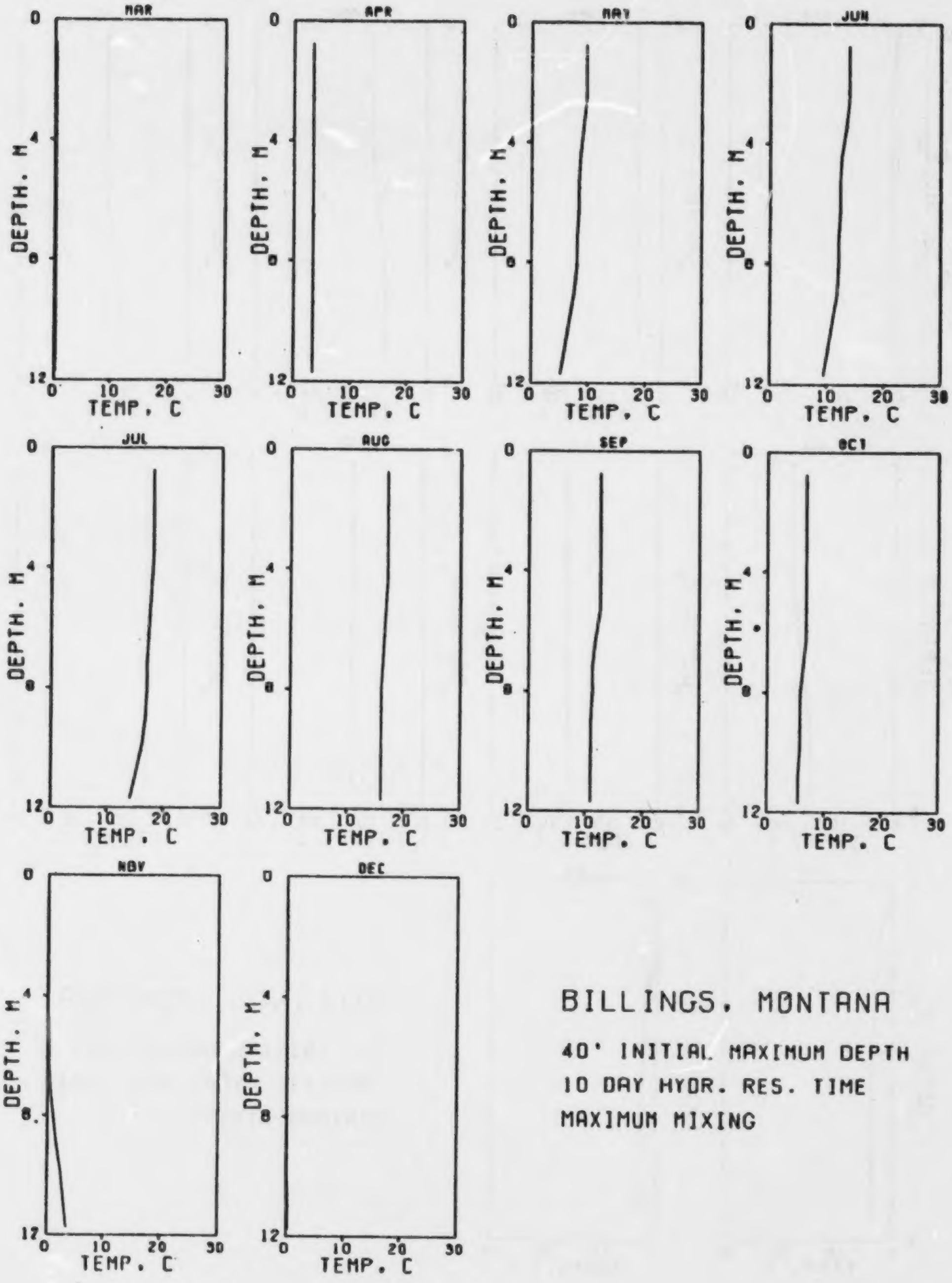
BILLINGS, MONTANA

20' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



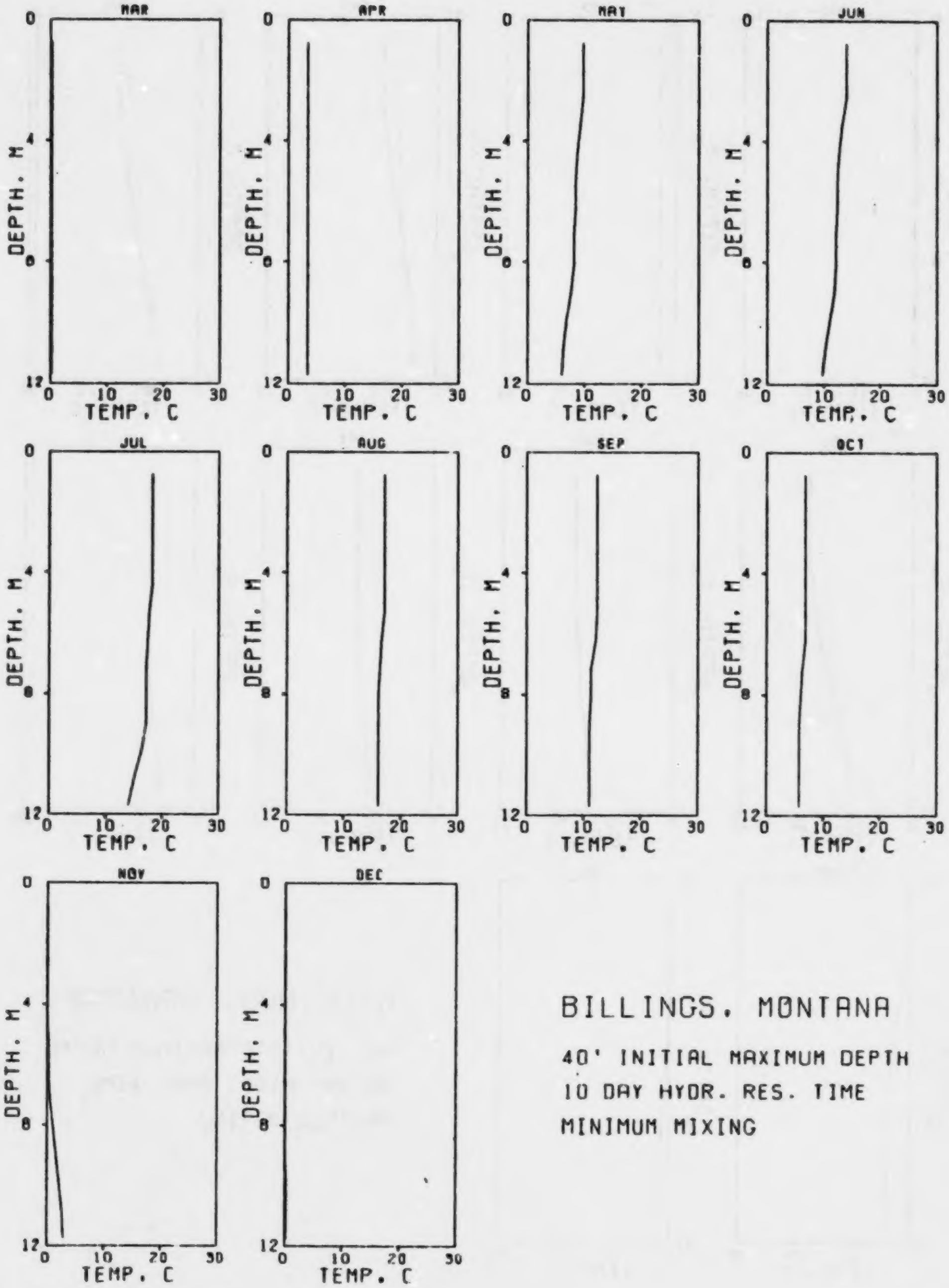
BILLINGS, MONTANA

20' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



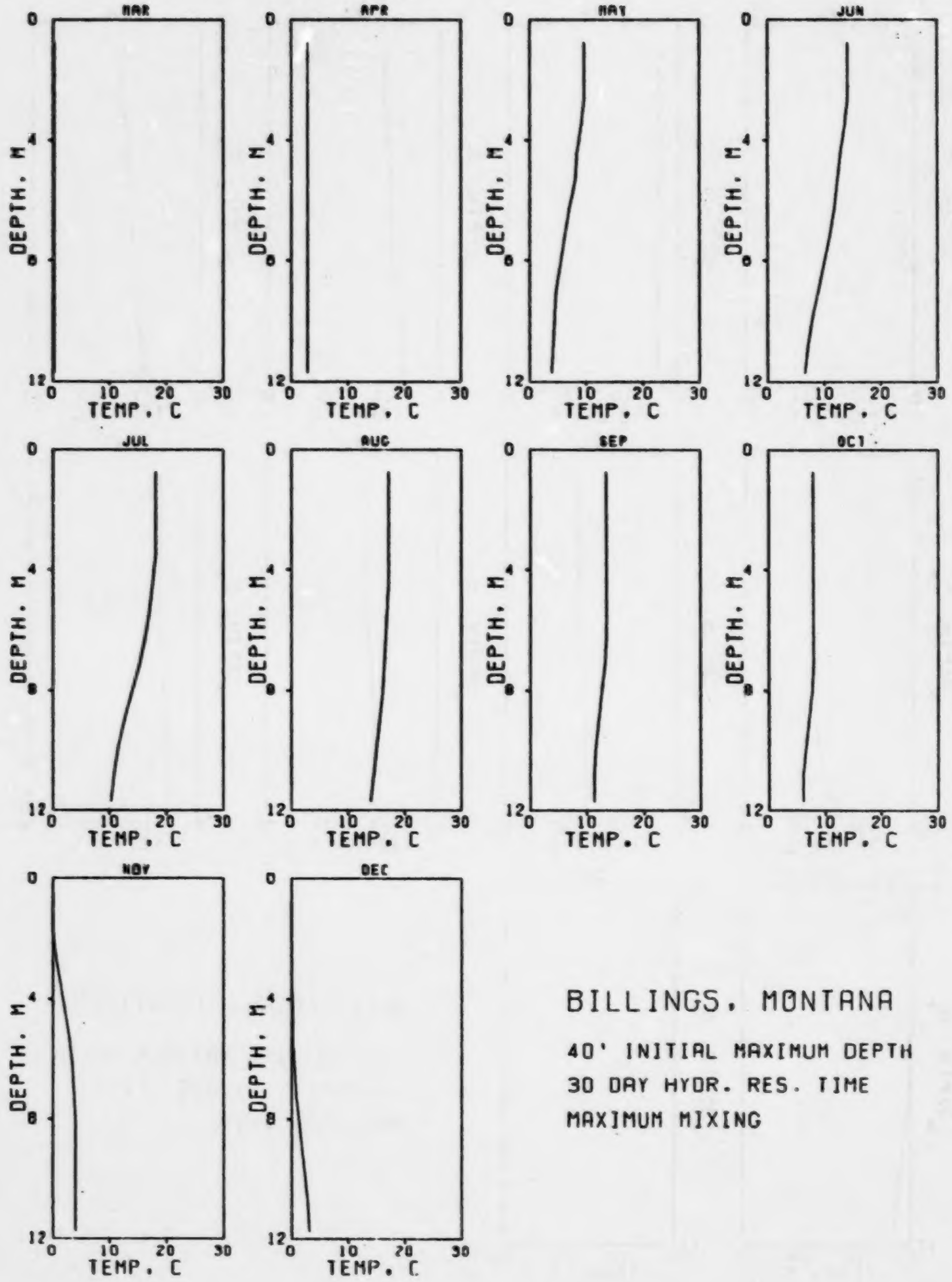
BILLINGS, MONTANA

40' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



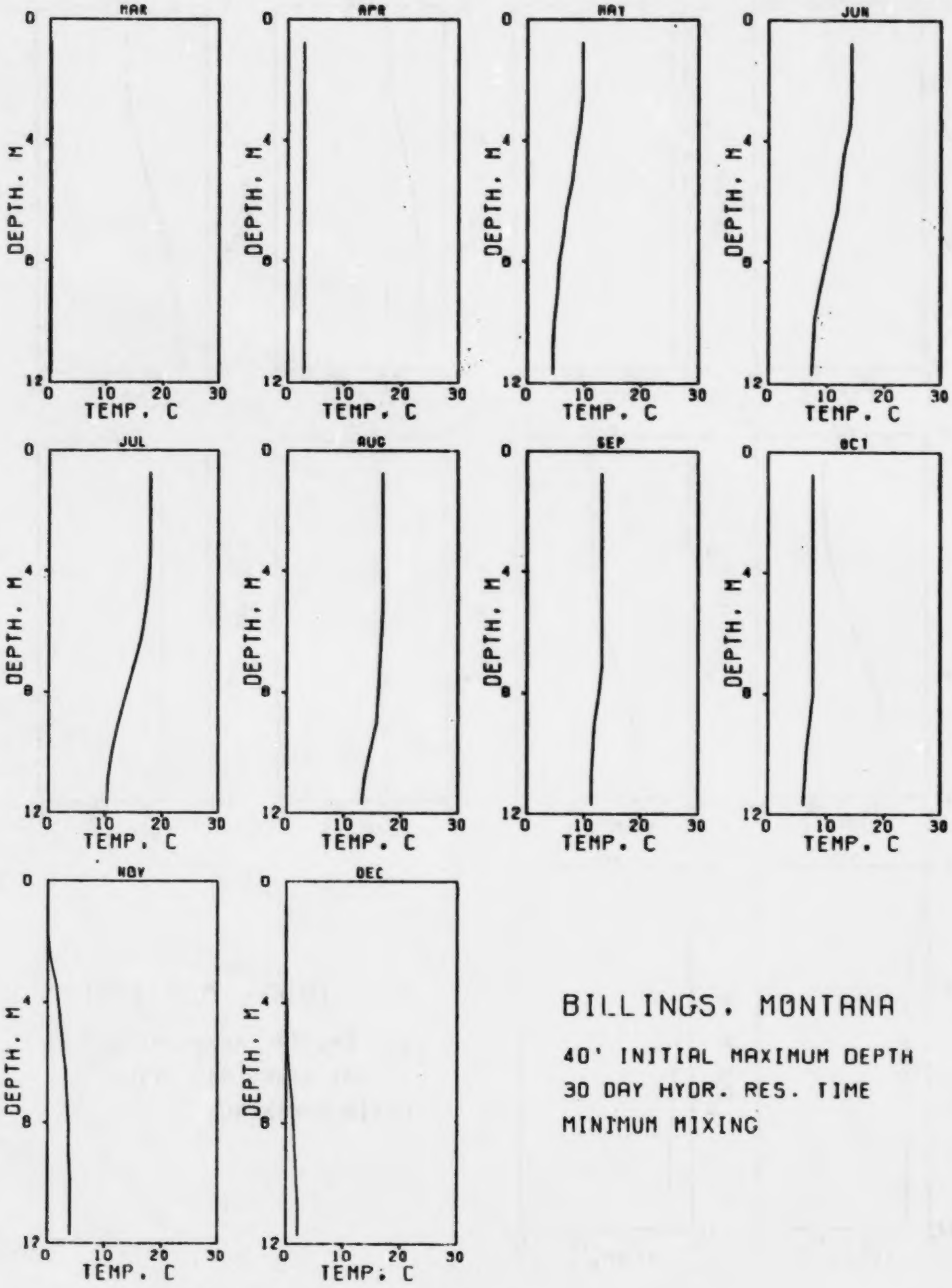
BILLINGS, MONTANA

40' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



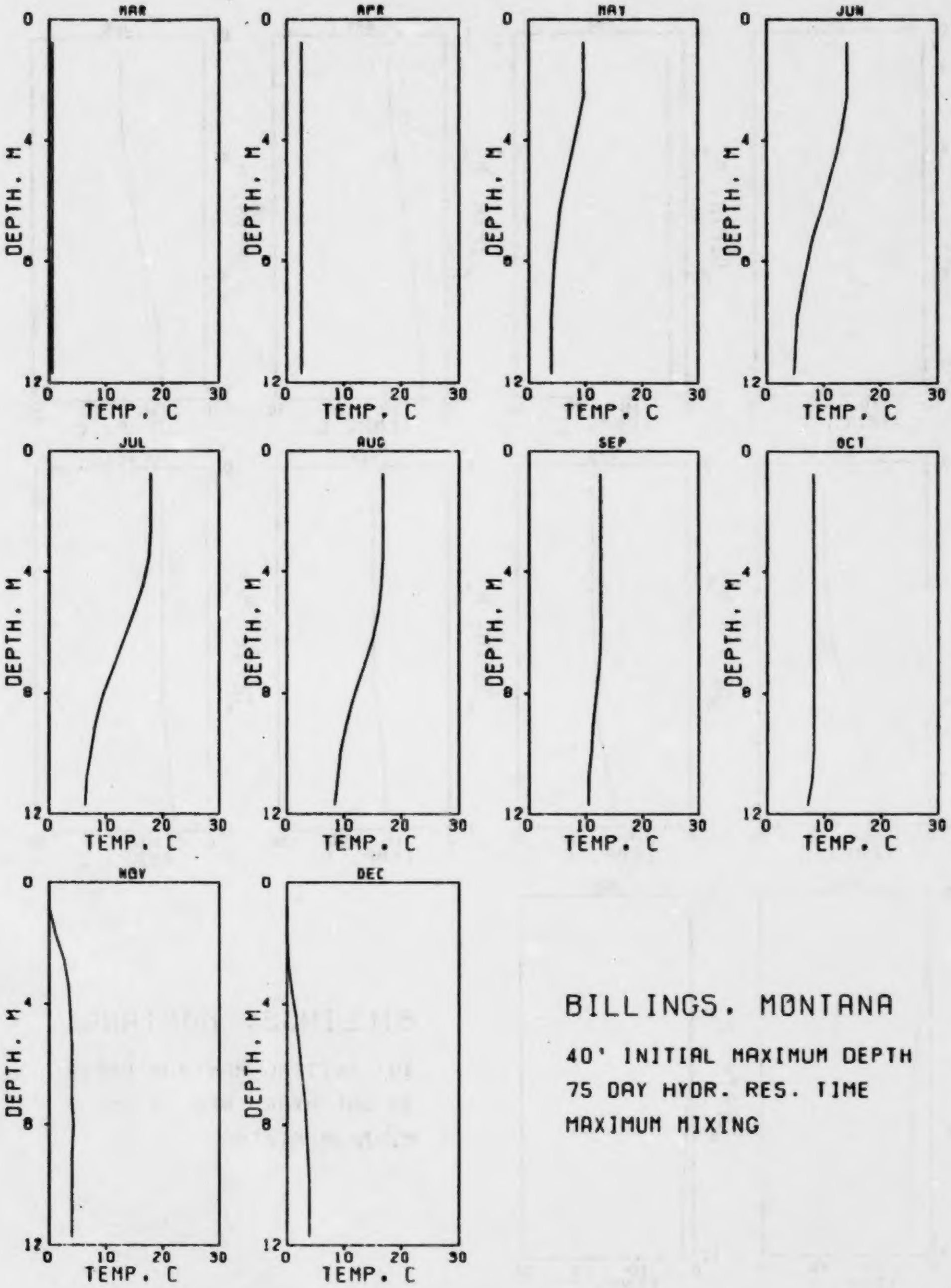
BILLINGS, MONTANA

40' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



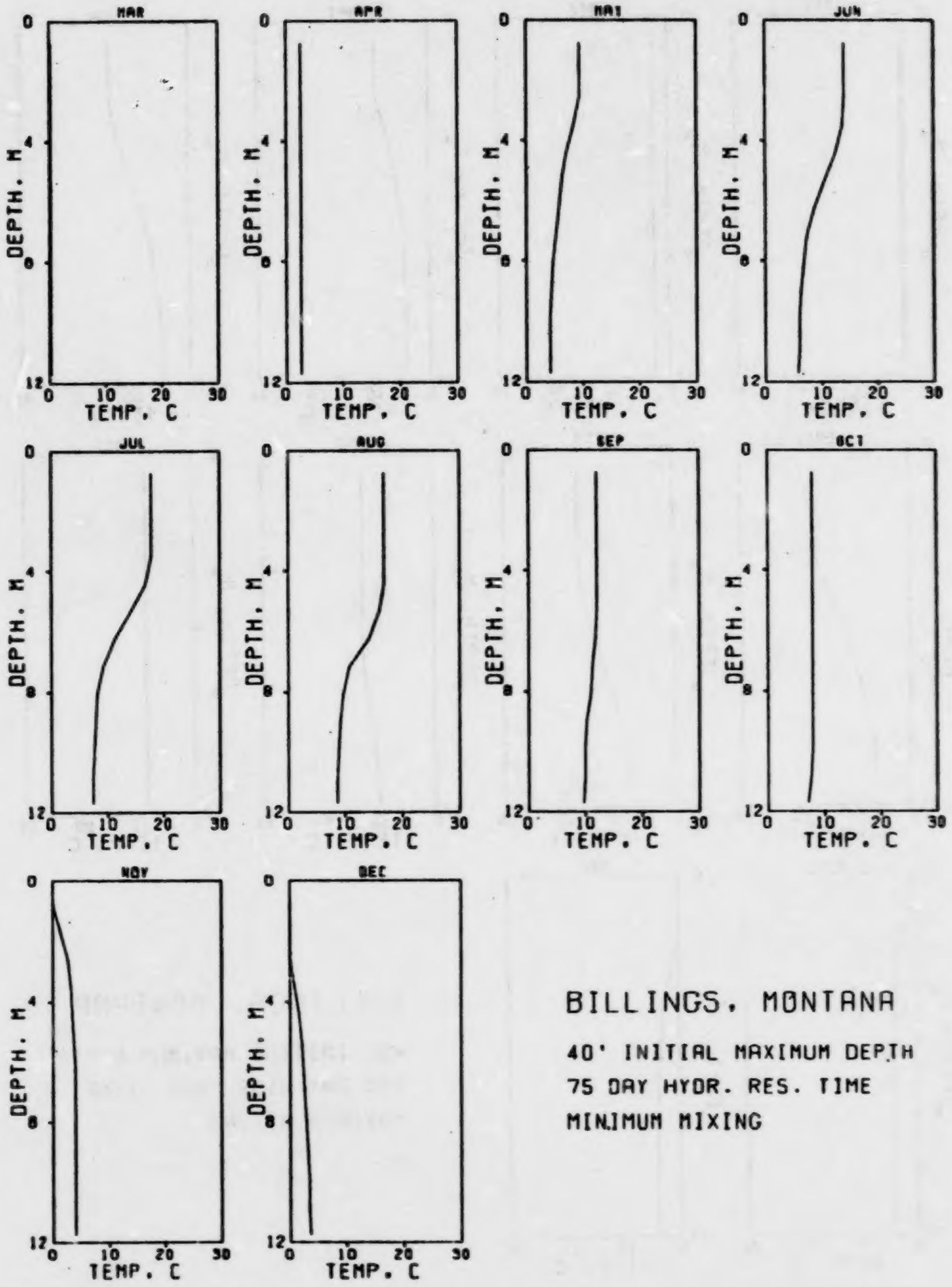
BILLINGS, MONTANA

40' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



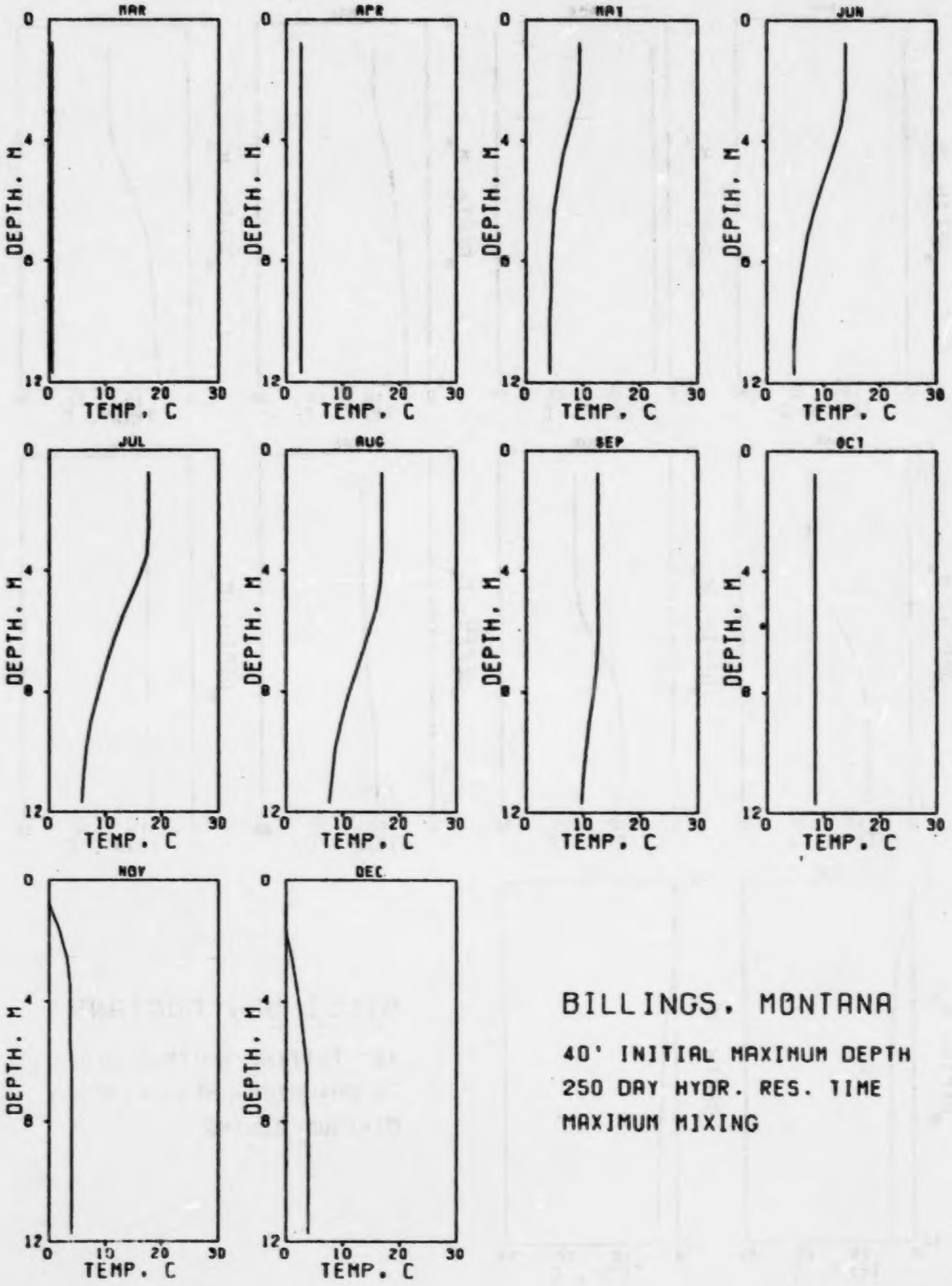
BILLINGS, MONTANA

40' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



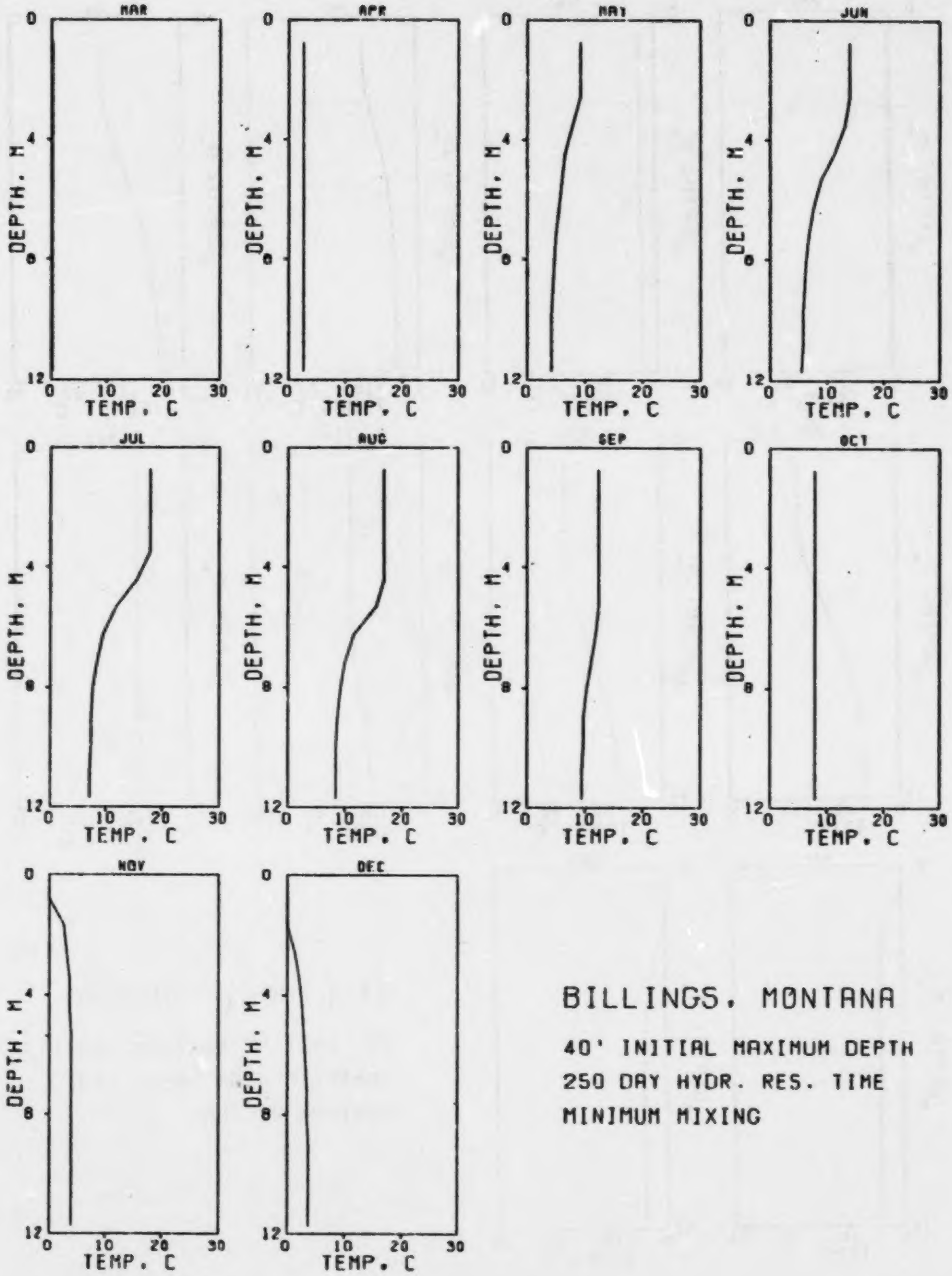
BILLINGS, MONTANA

40' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



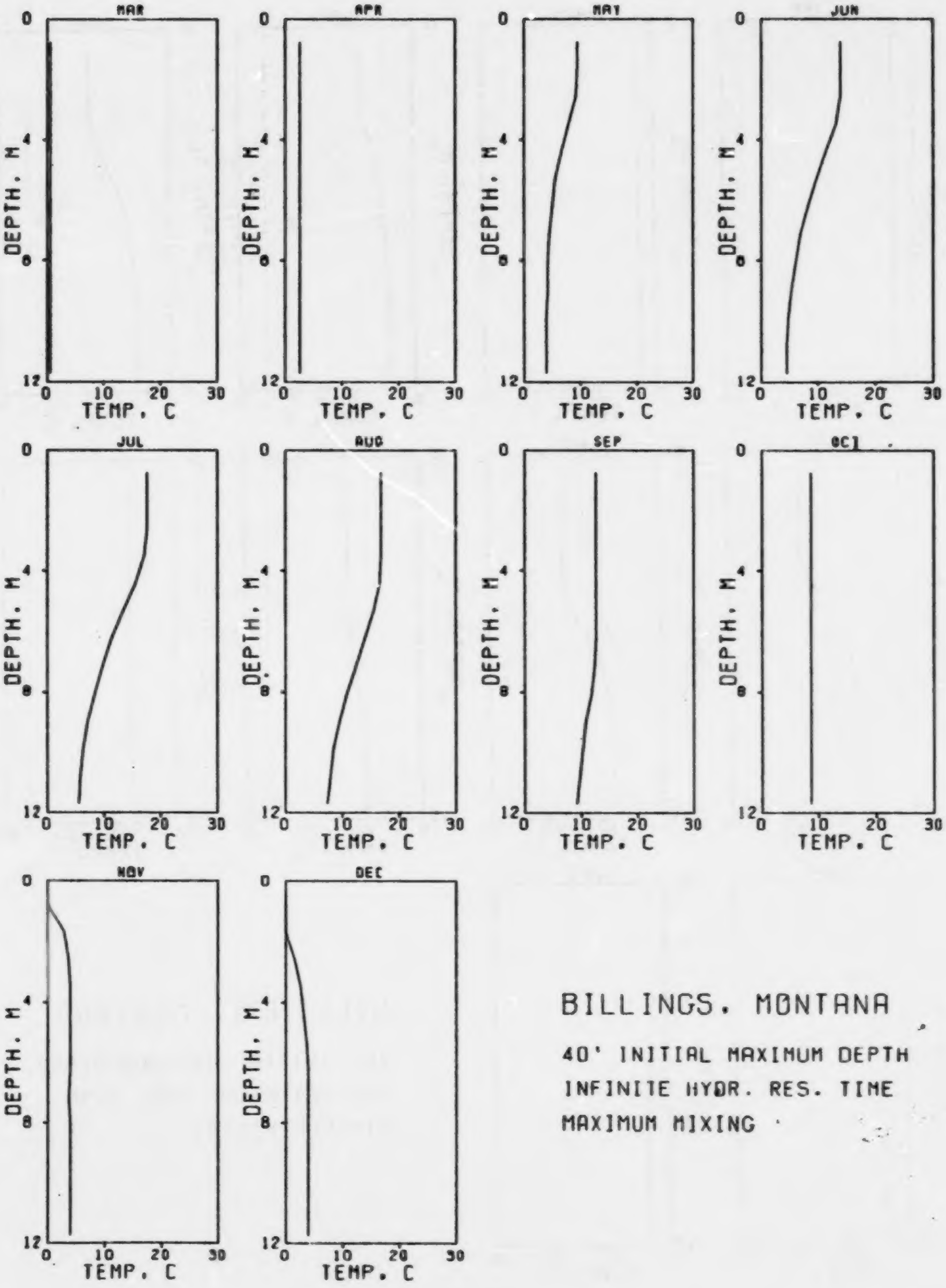
BILLINGS, MONTANA

40' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



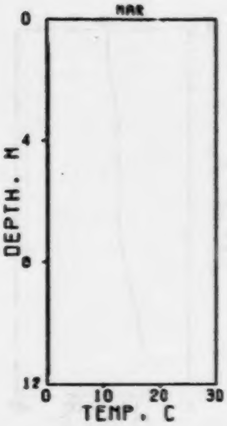
BILLINGS, MONTANA

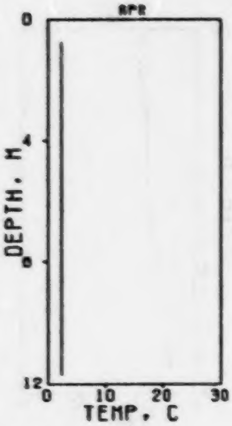
40' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING

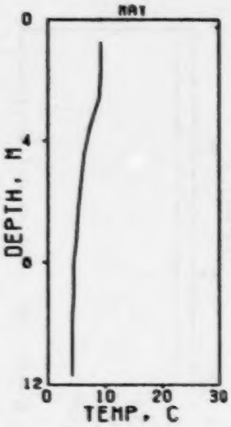


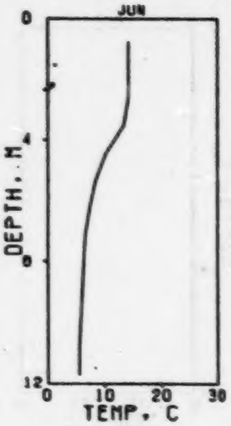
BILLINGS, MONTANA

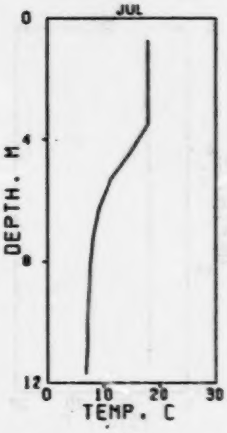
40' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING

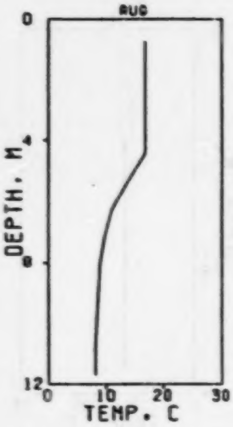


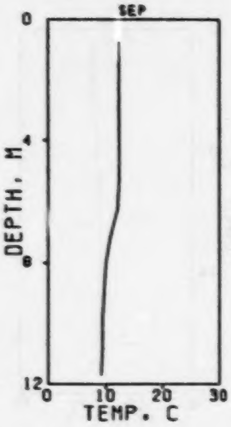


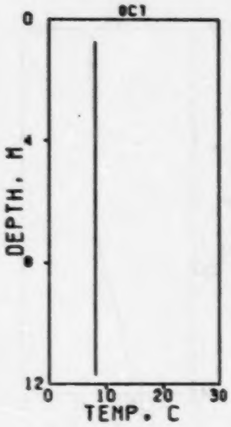


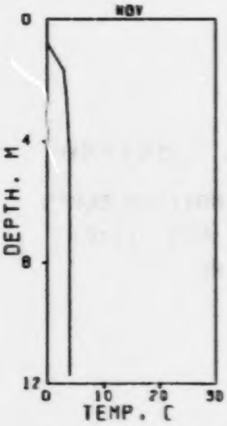


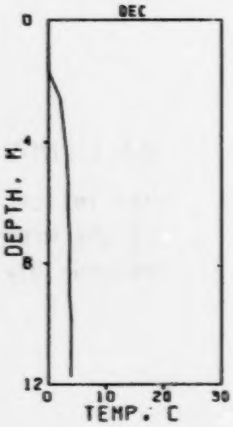






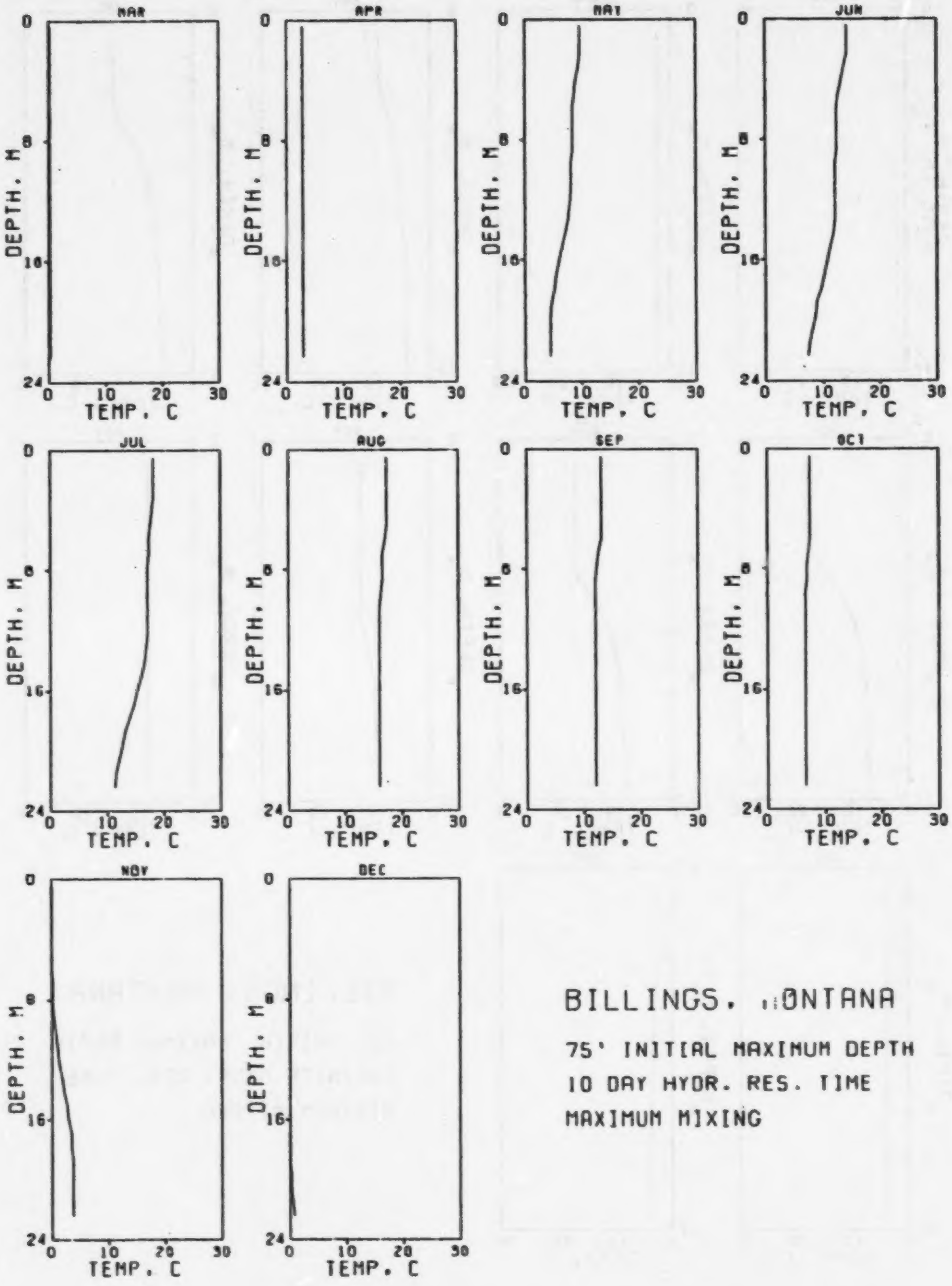






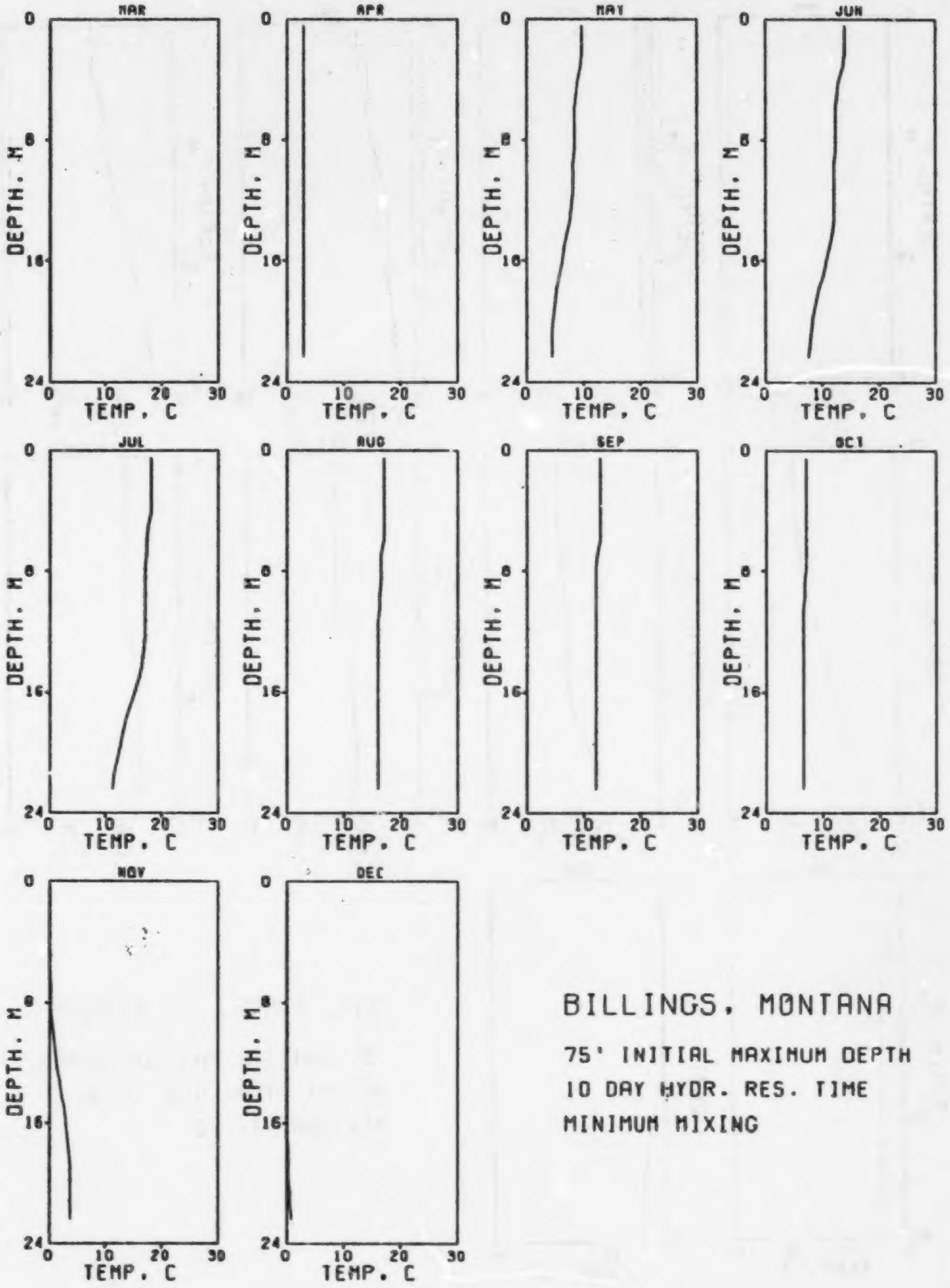
BILLINGS, MONTANA

40' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



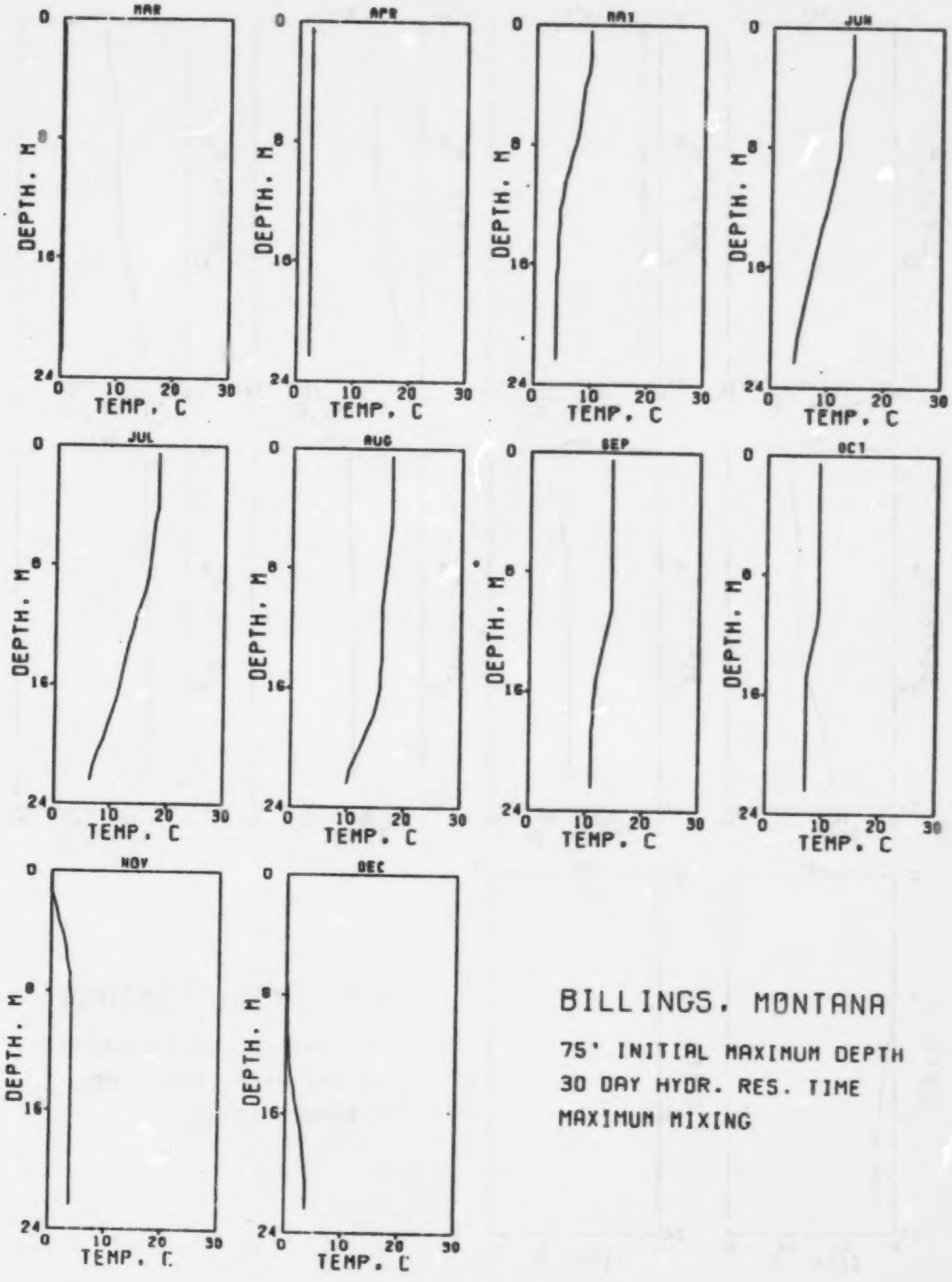
BILLINGS, MONTANA

75' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



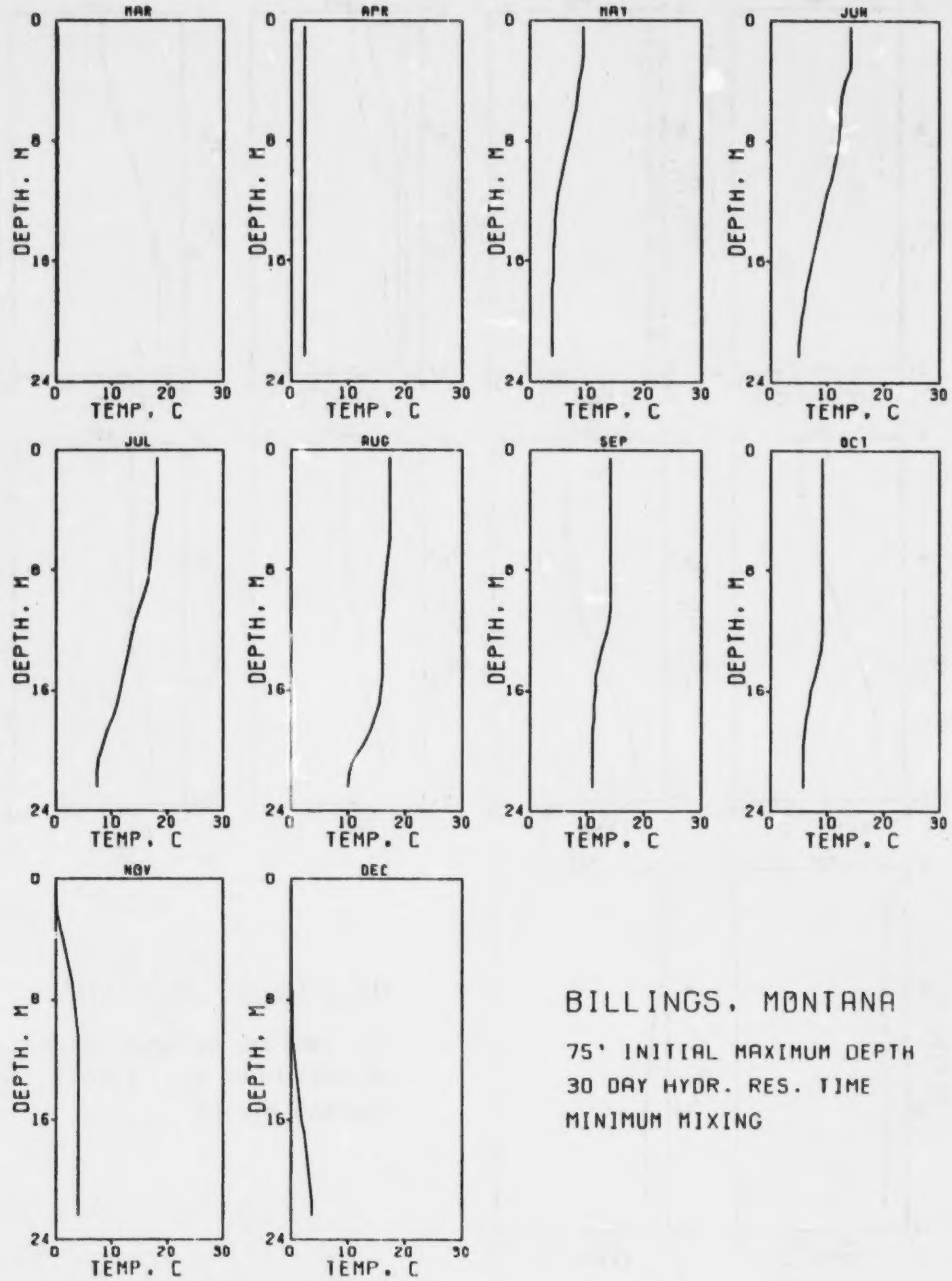
BILLINGS, MONTANA

75' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



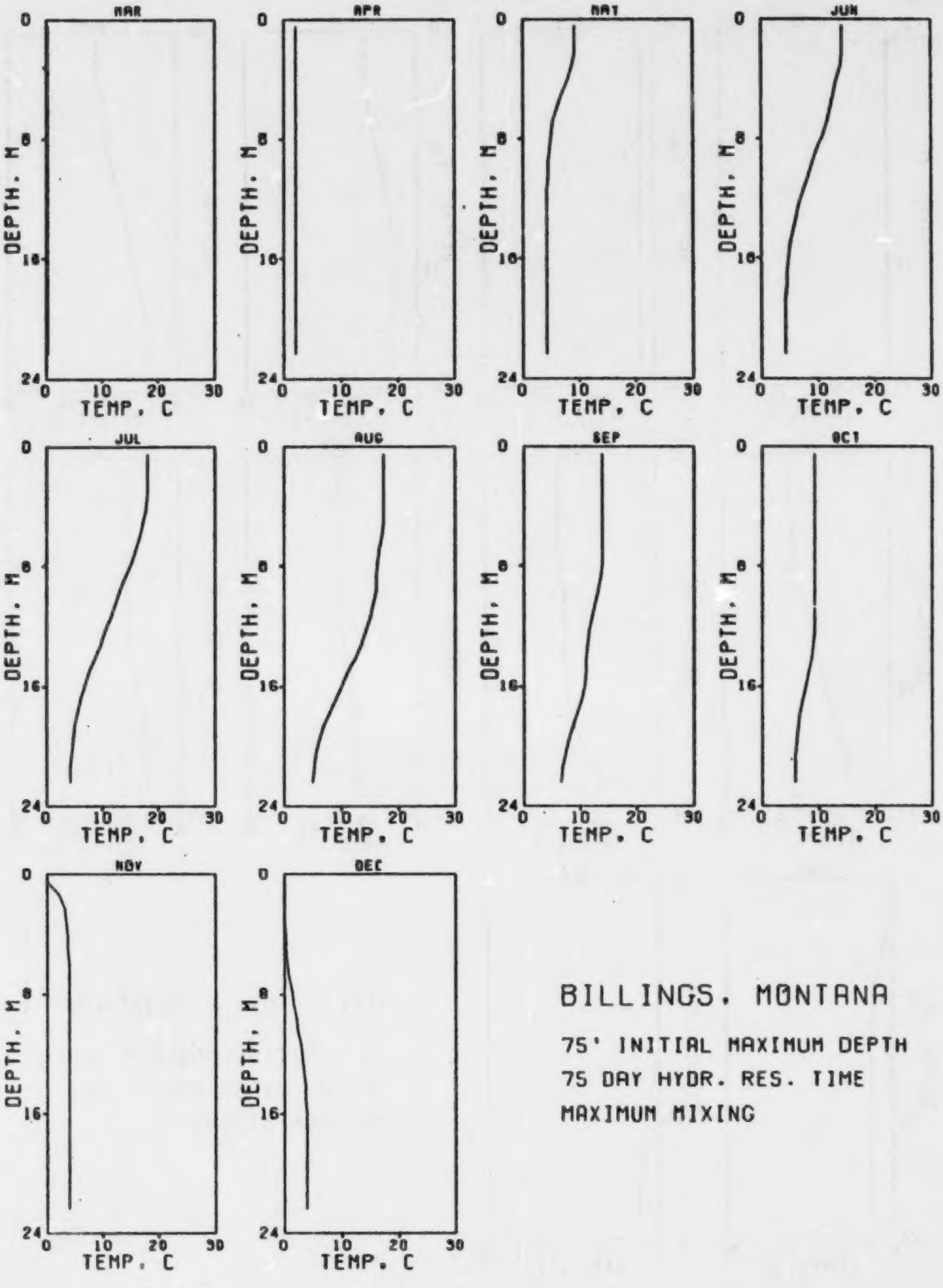
BILLINGS, MONTANA

75' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



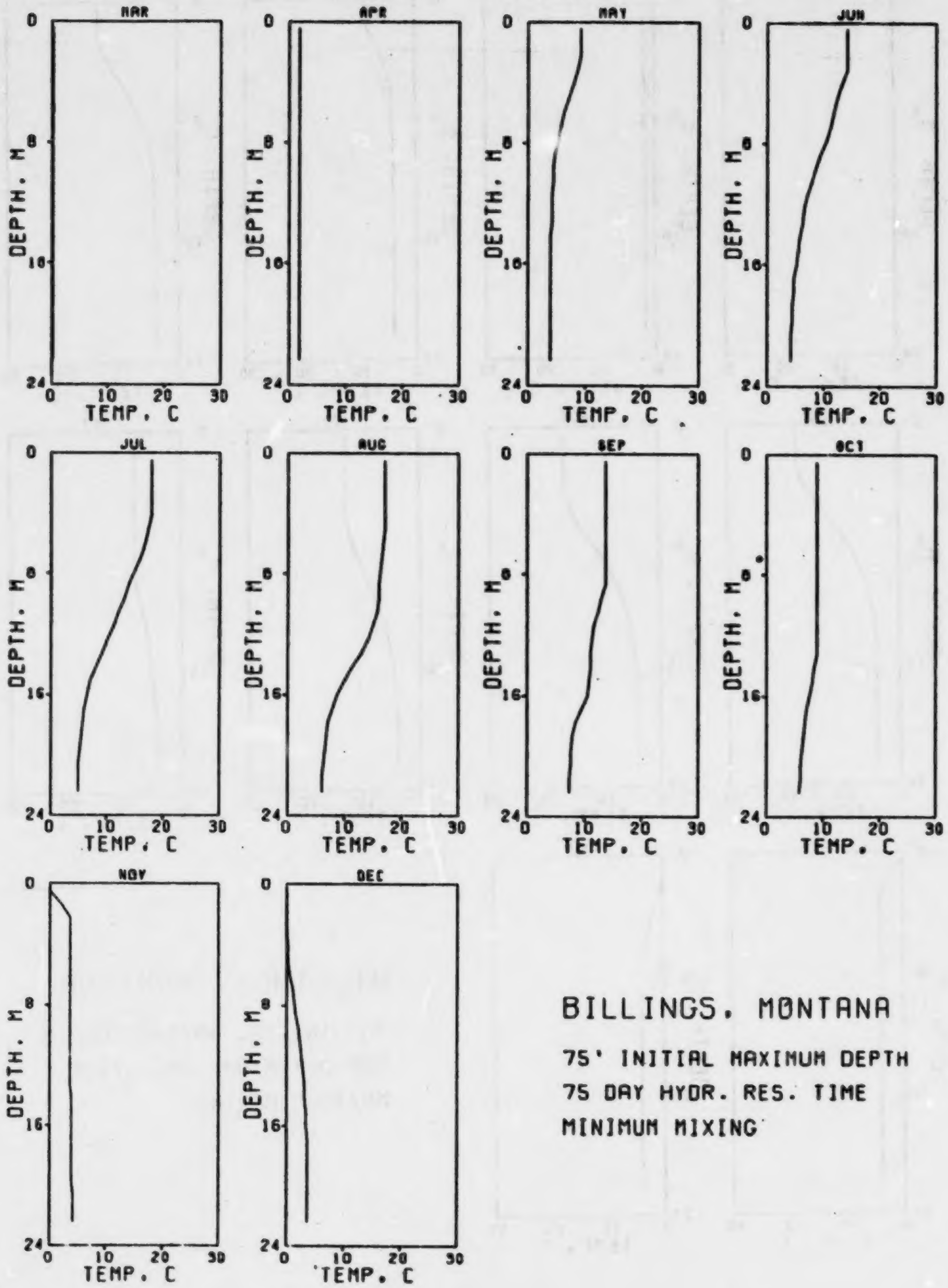
BILLINGS, MONTANA

75' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



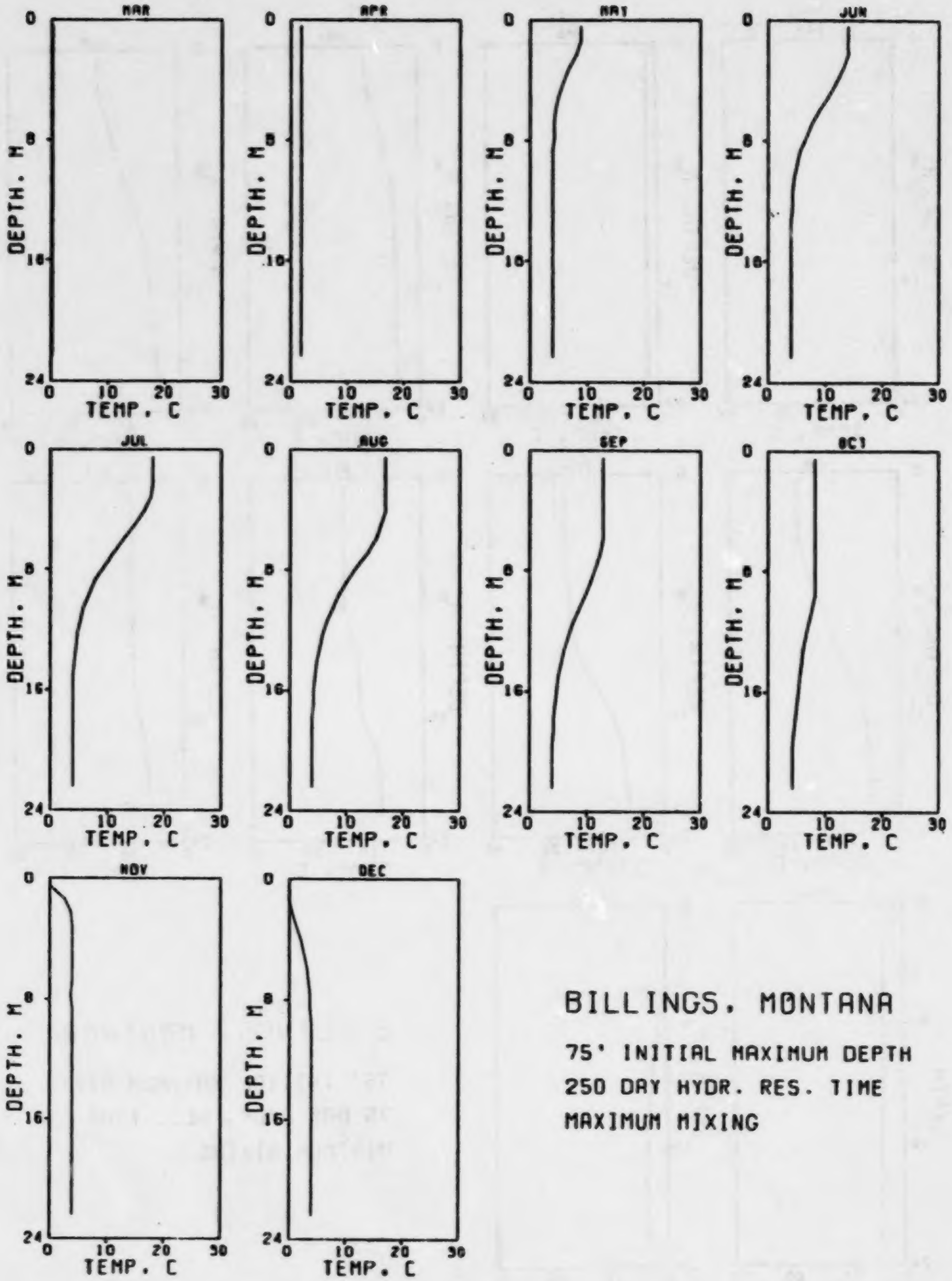
BILLINGS, MONTANA

75' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



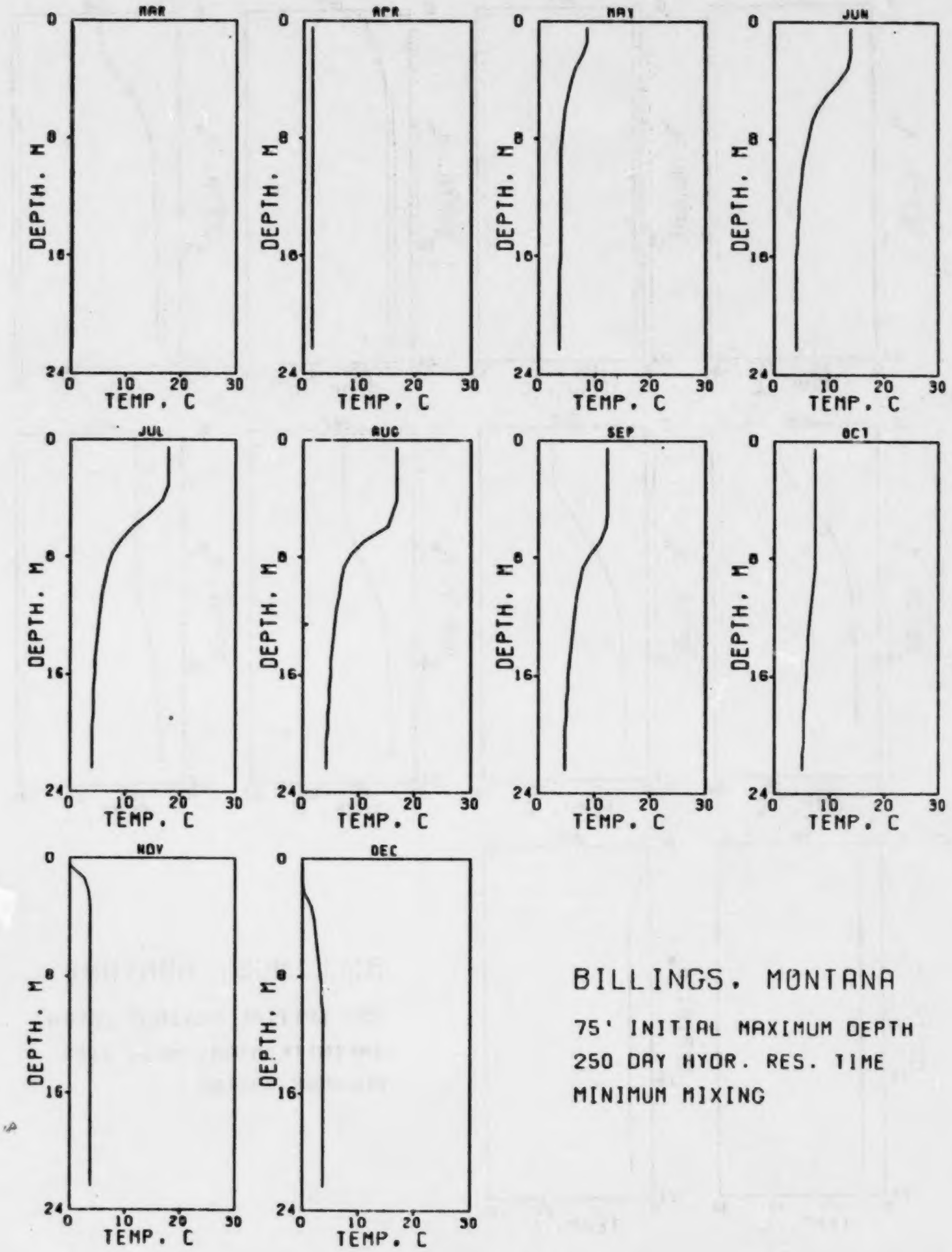
BILLINGS, MONTANA

75' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



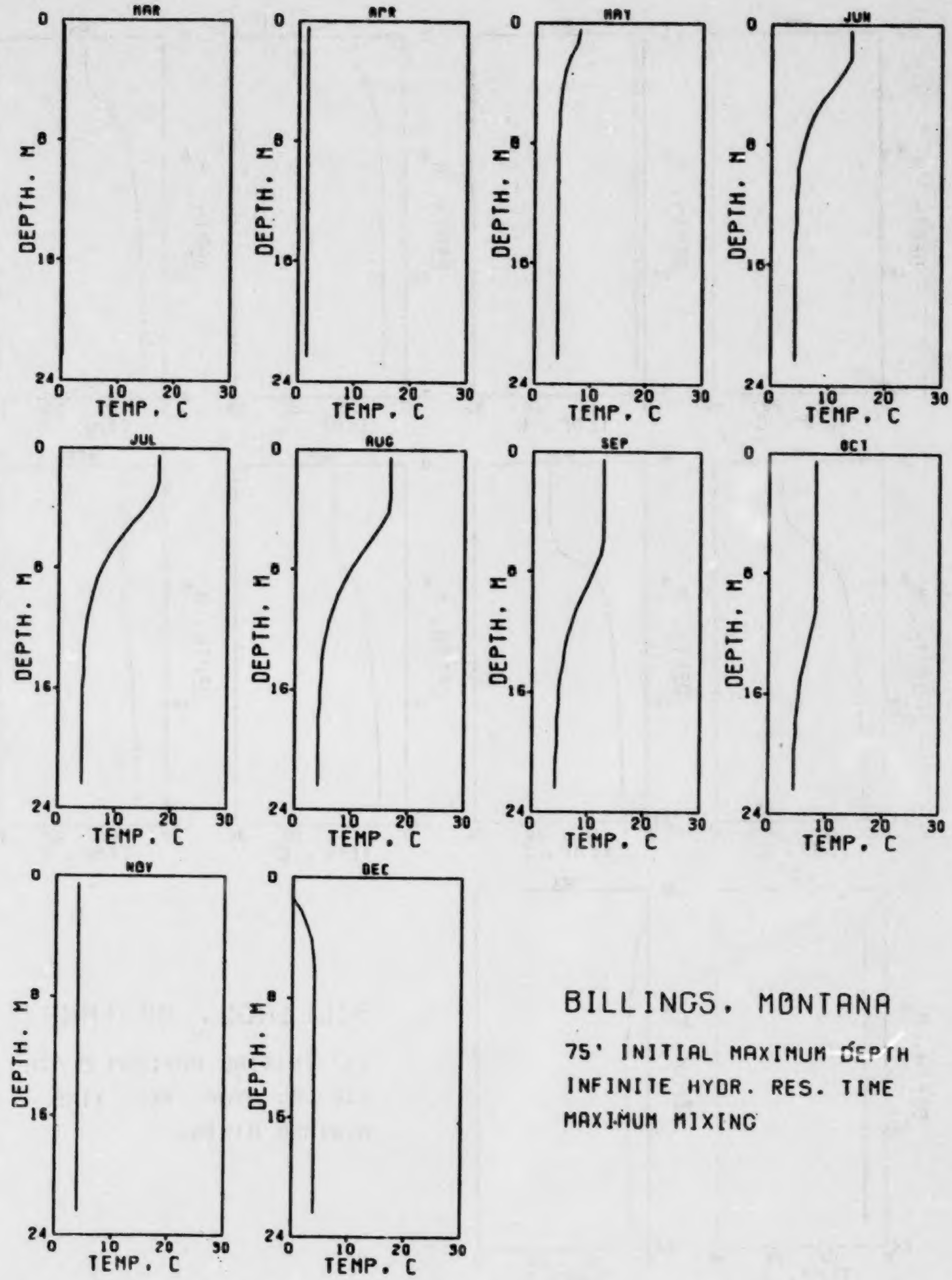
BILLINGS, MONTANA

75' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



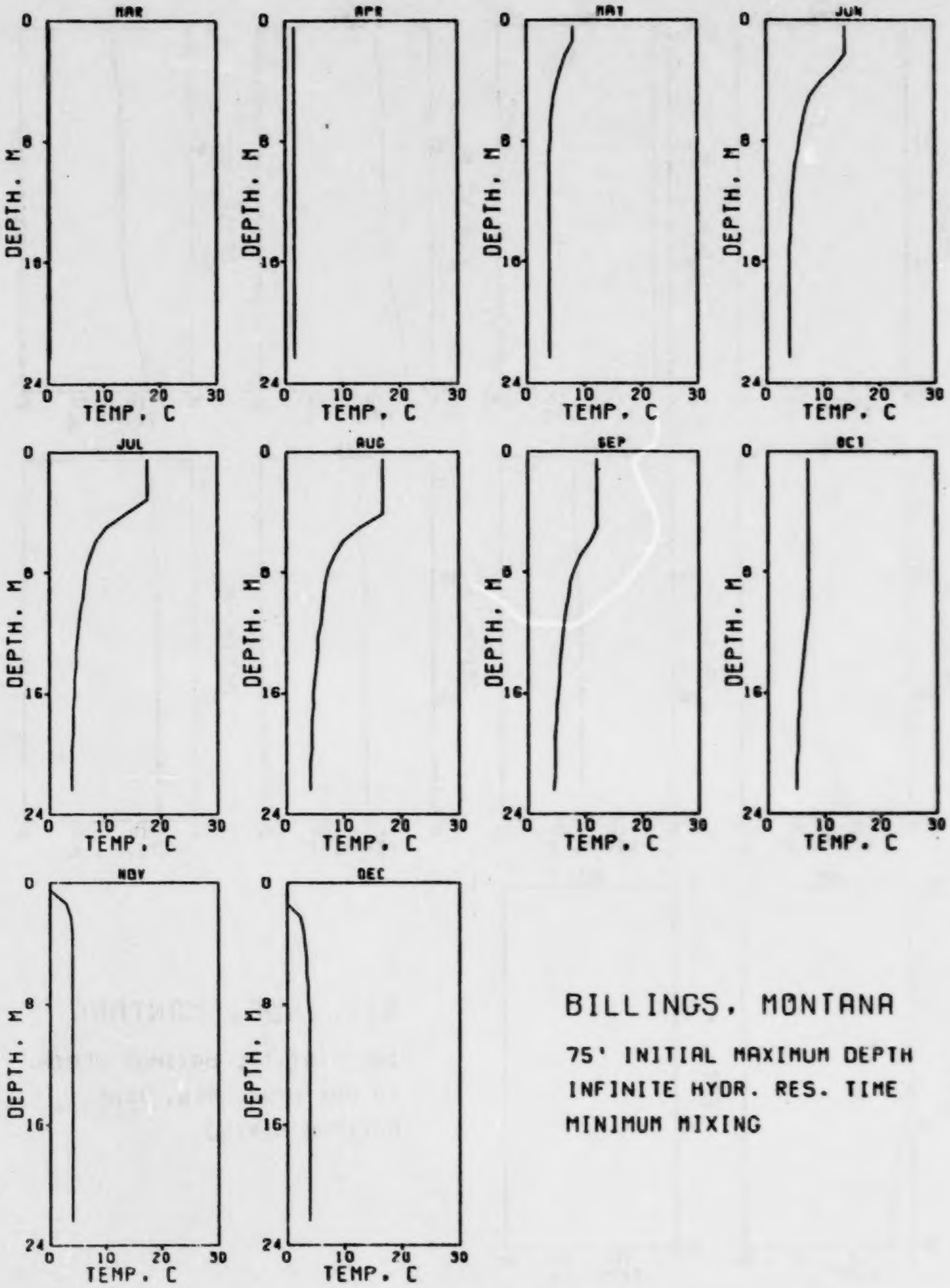
BILLINGS, MONTANA

75' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



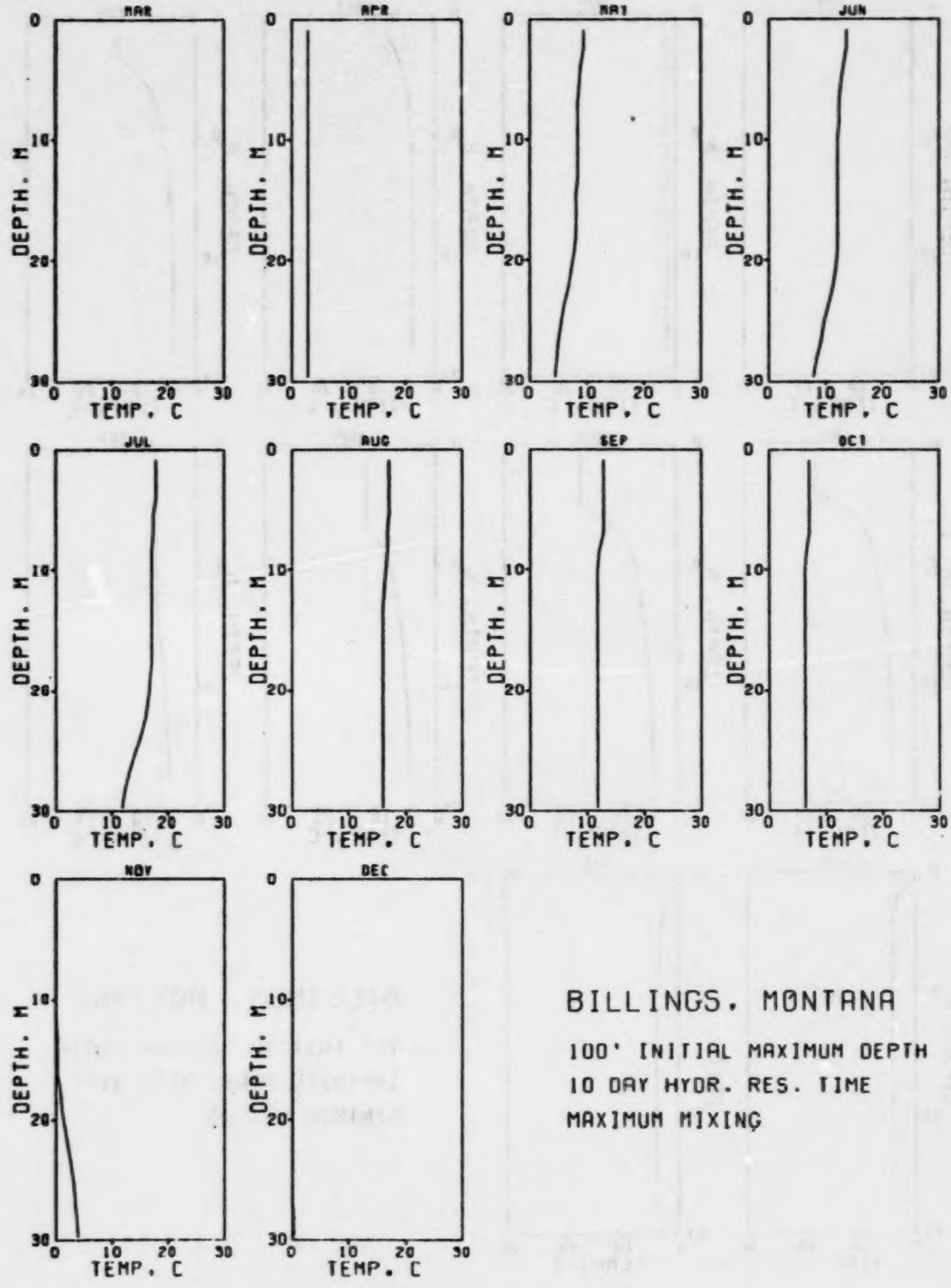
BILLINGS, MONTANA

75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



BILLINGS, MONTANA

75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING

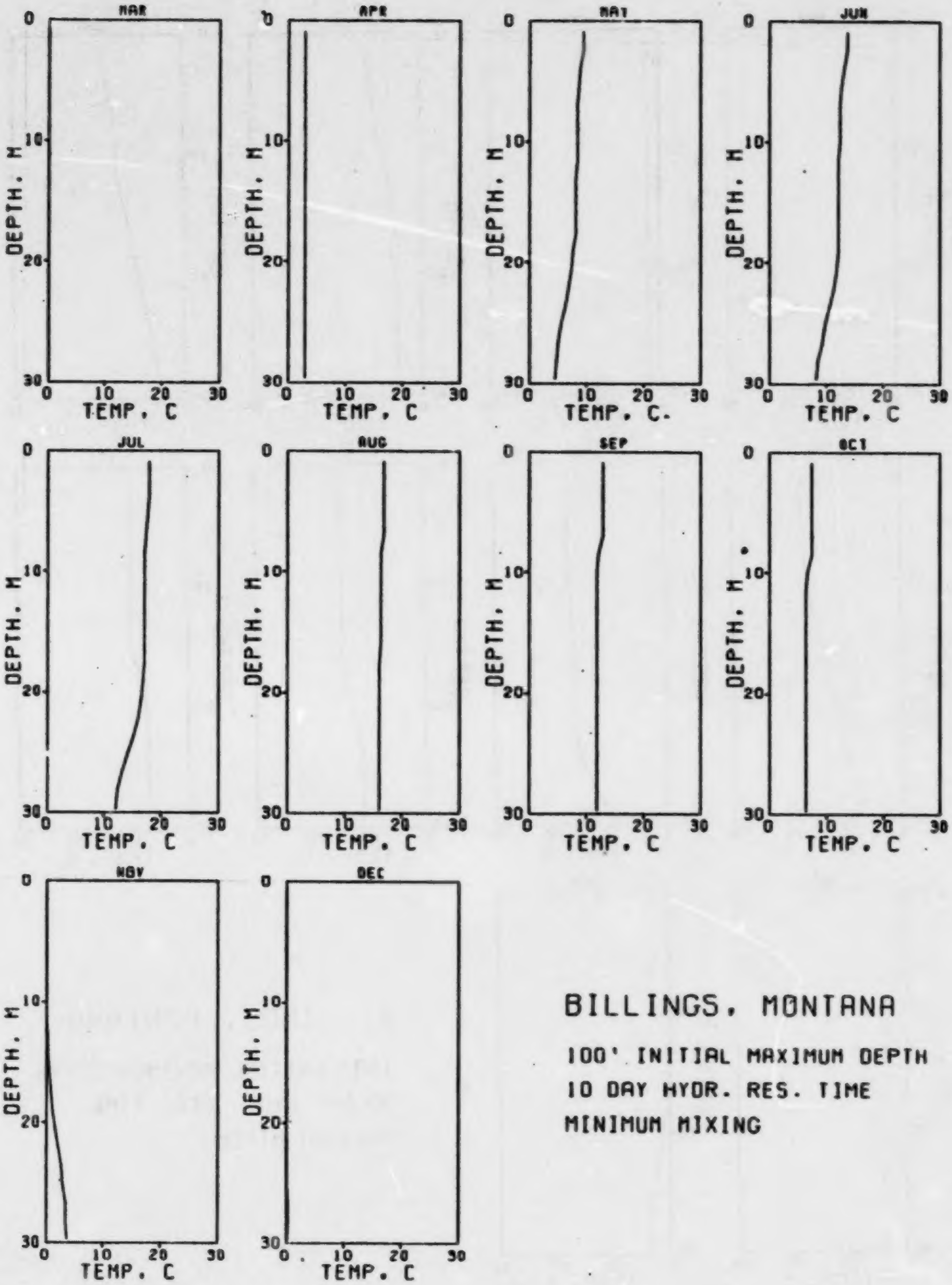


BILLINGS, MONTANA

100' INITIAL MAXIMUM DEPTH

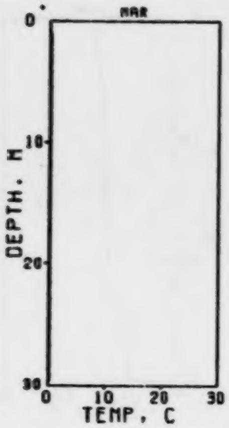
10 DAY HYDR. RES. TIME

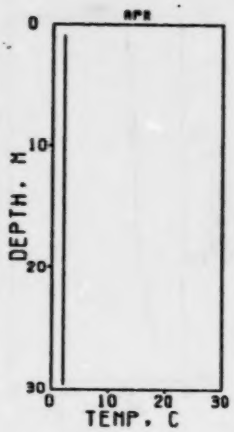
MAXIMUM MIXING

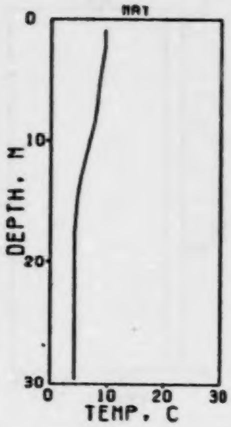


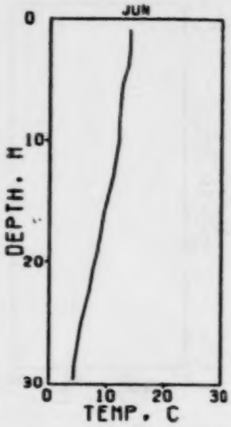
BILLINGS, MONTANA

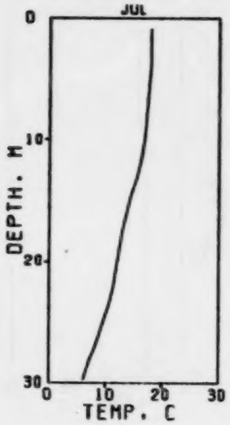
100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING

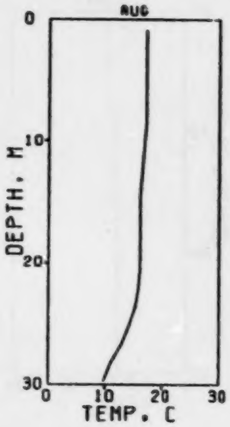


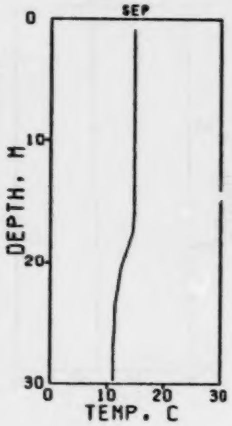


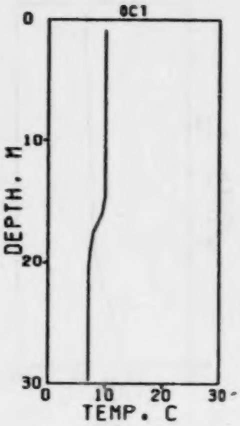


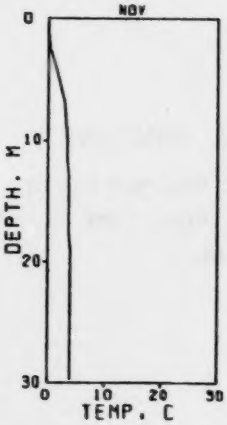


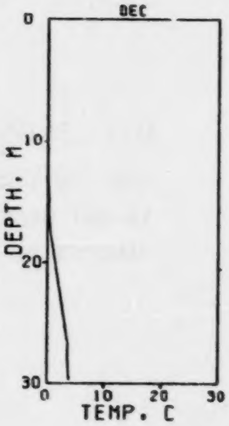






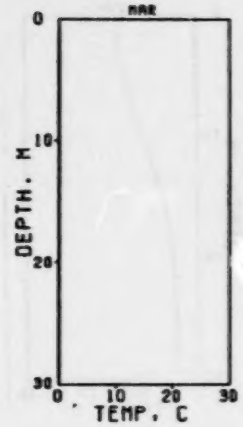


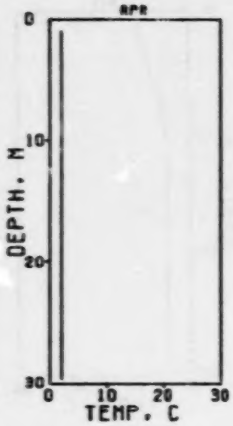


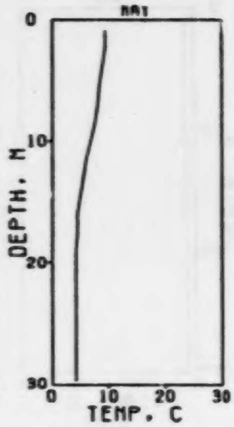


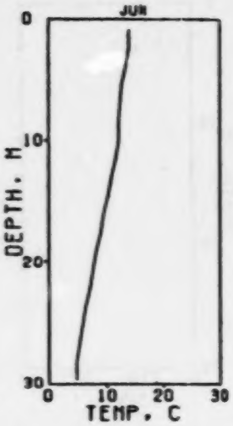
BILLINGS, MONTANA

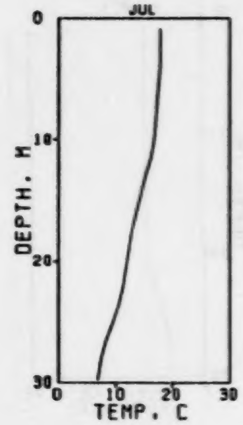
100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING

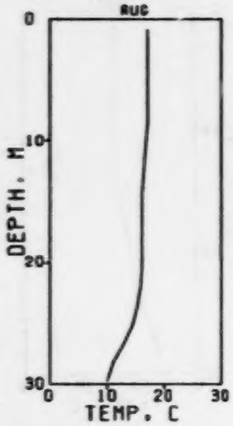


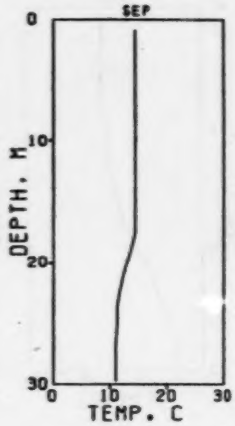


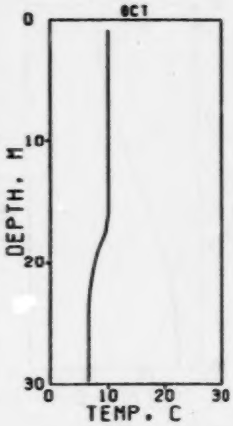


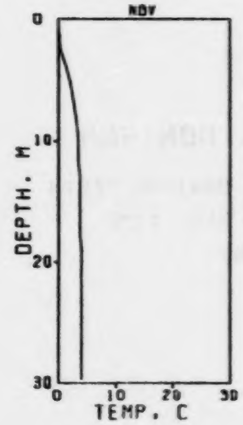


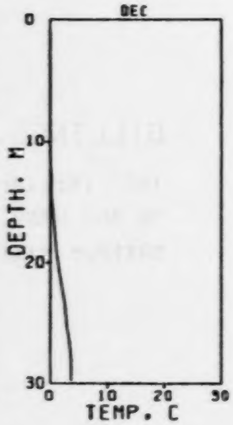






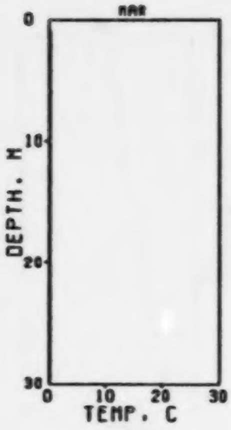


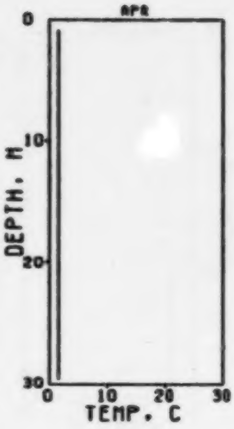


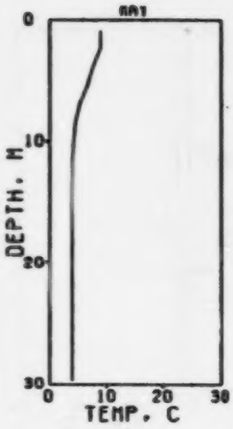


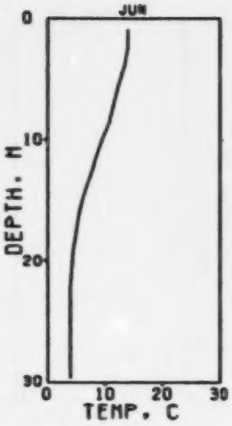
BILLINGS, MONTANA

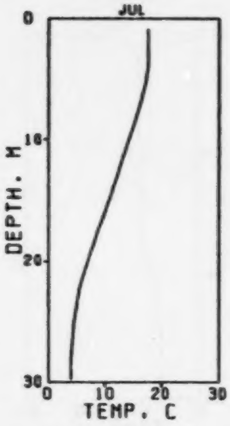
100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING

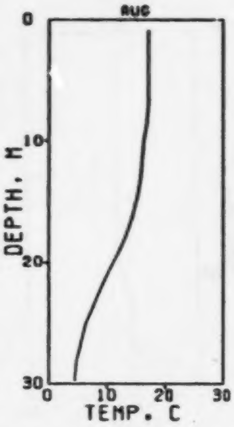


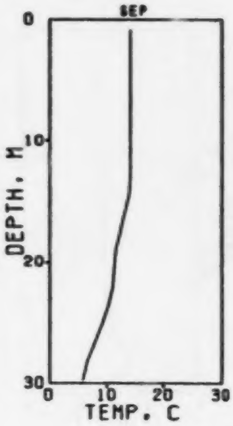


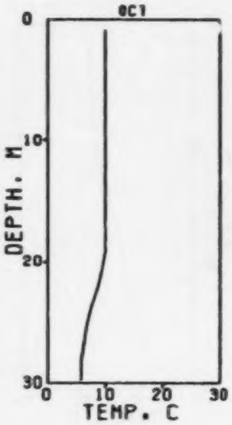


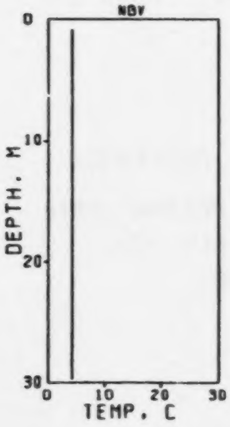


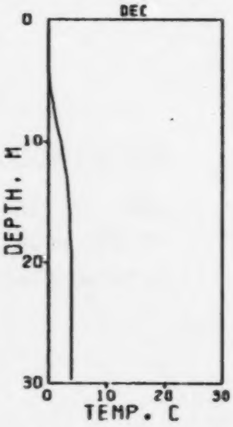






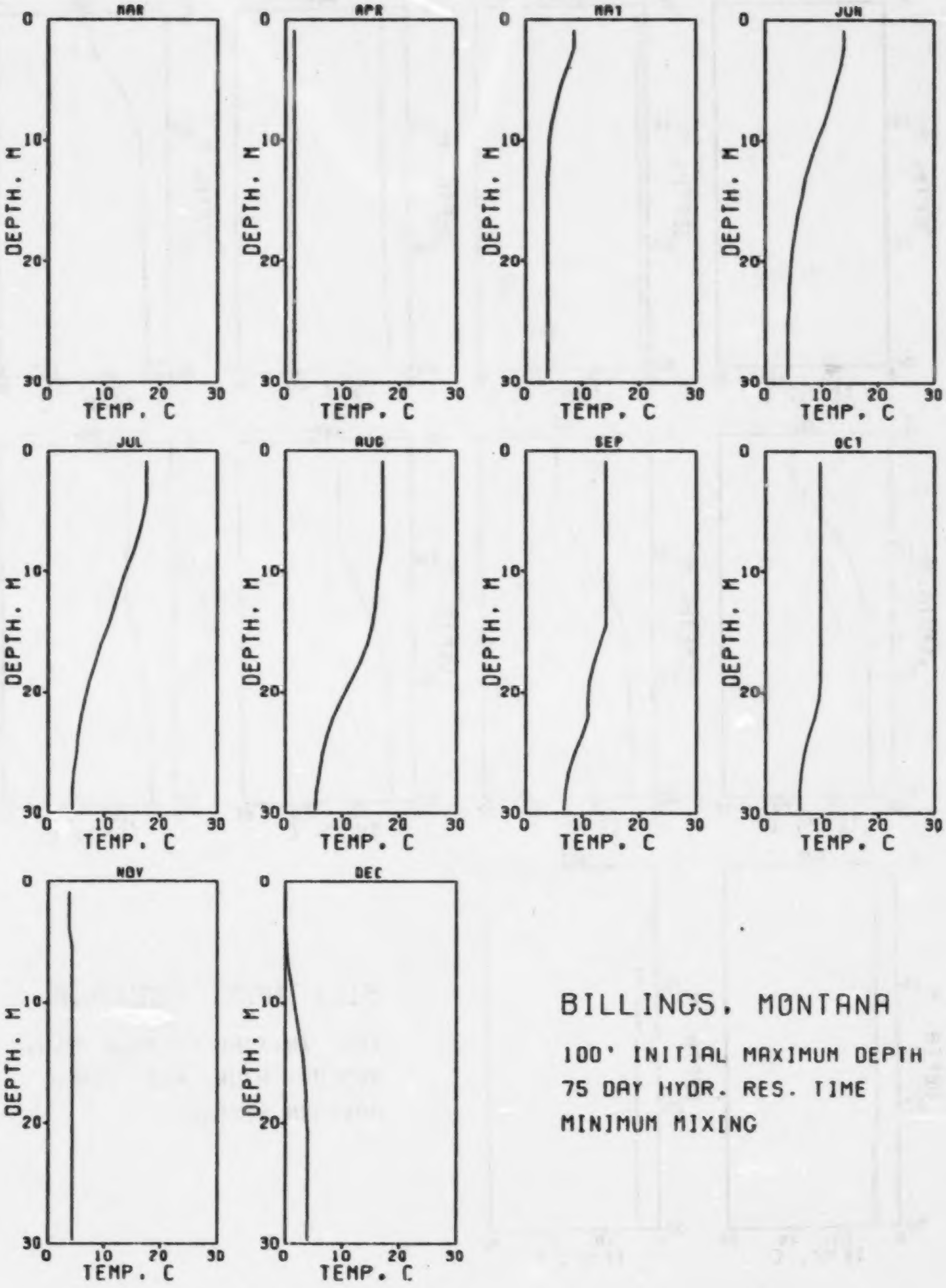






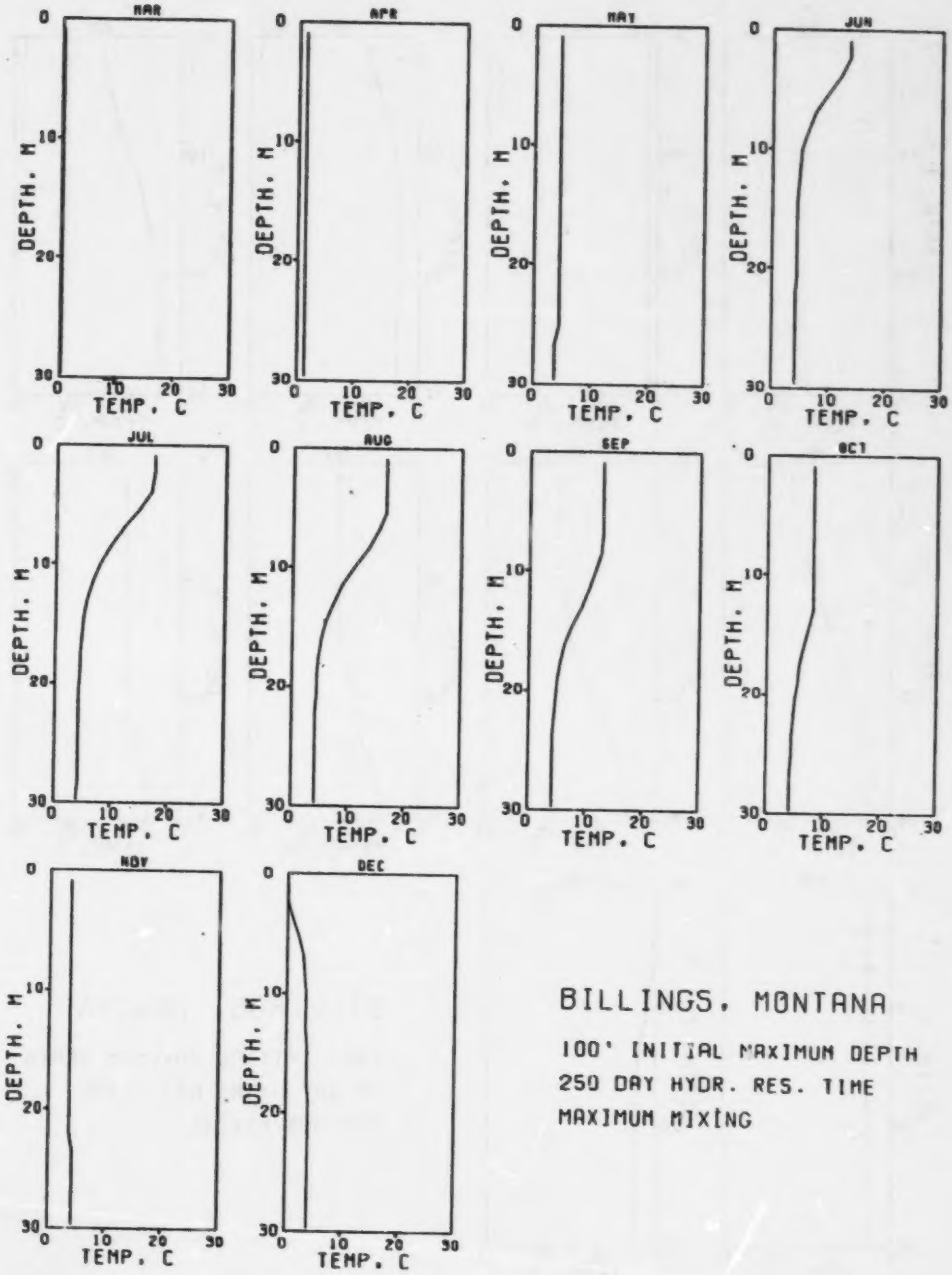
BILLINGS, MONTANA

100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



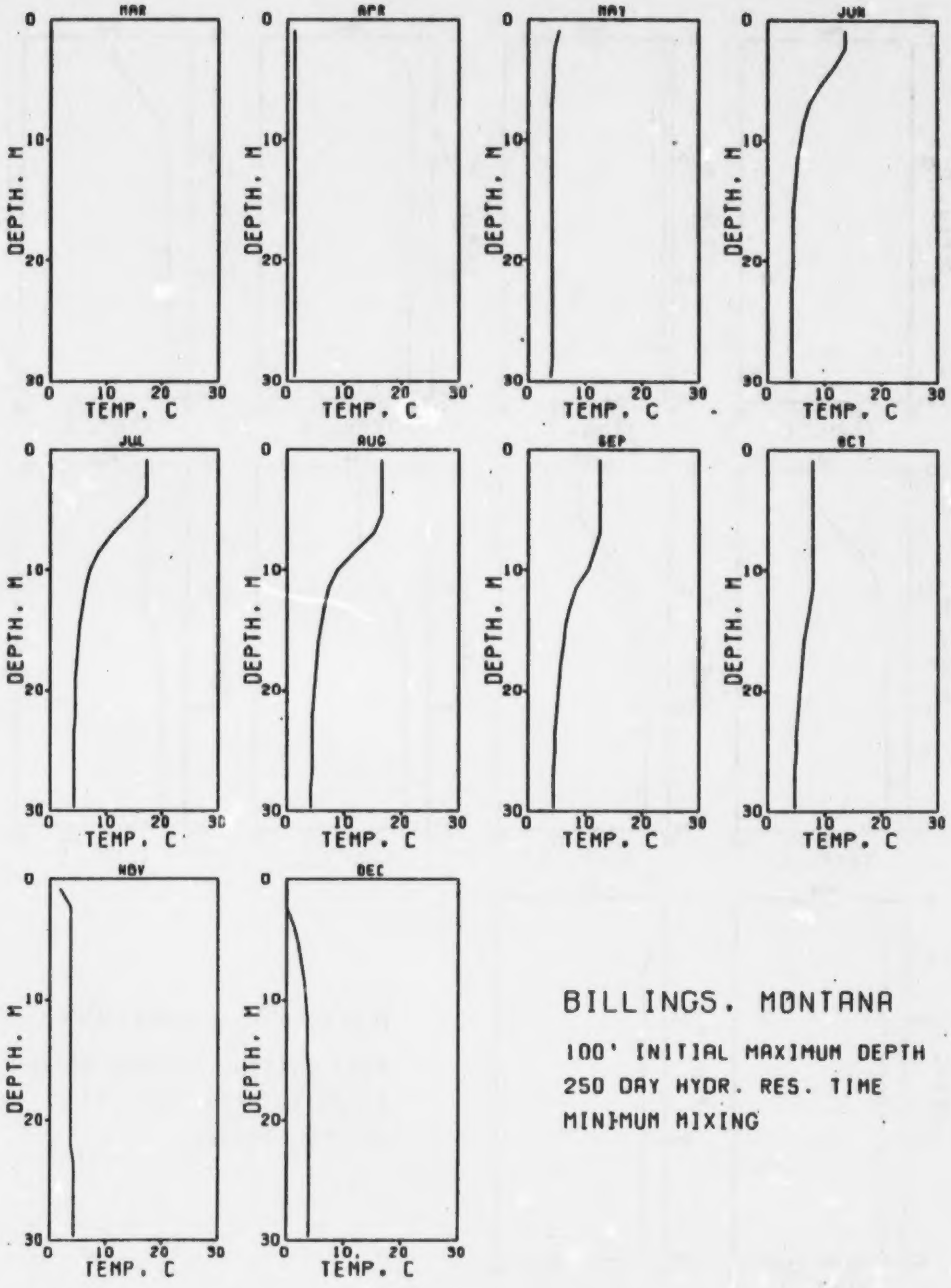
BILLINGS, MONTANA

100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



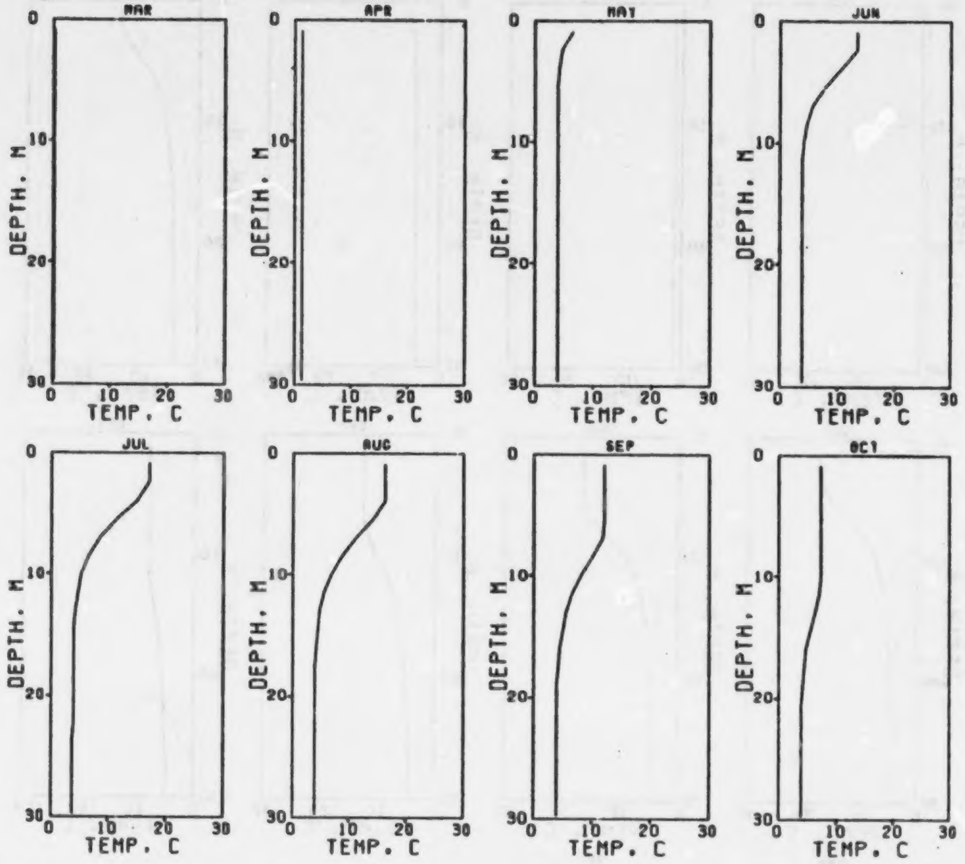
BILLINGS, MONTANA

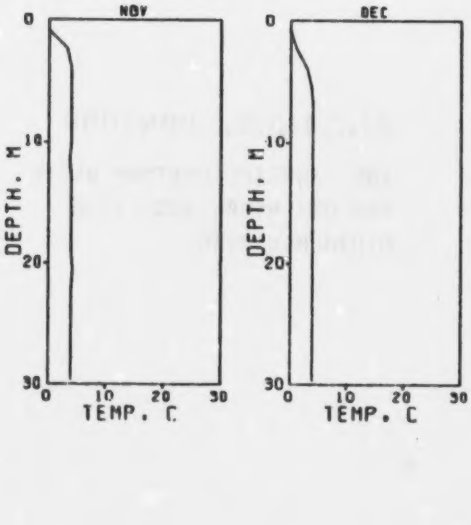
100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



BILLINGS, MONTANA

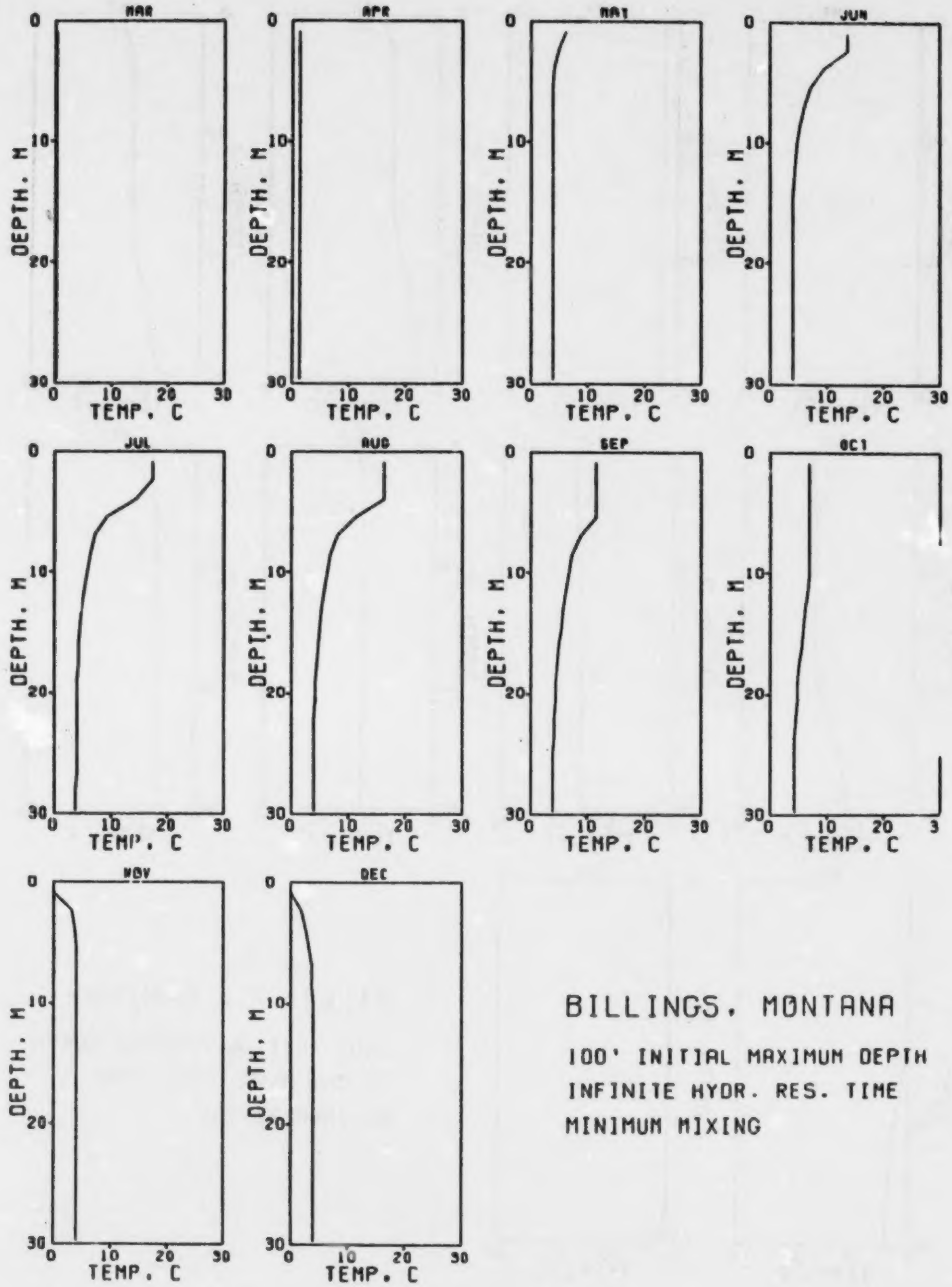
100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING





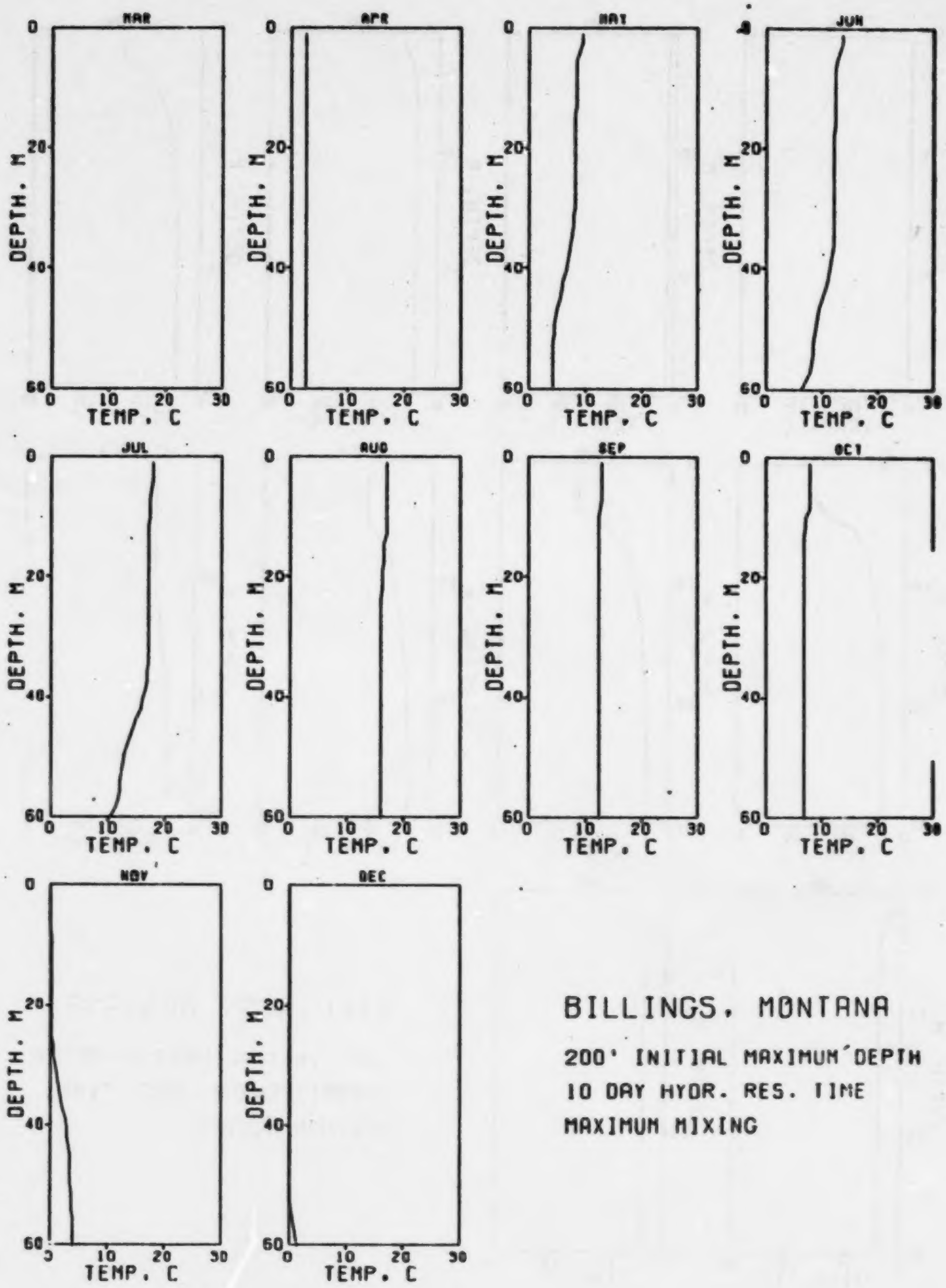
BILLINGS, MONTANA

100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



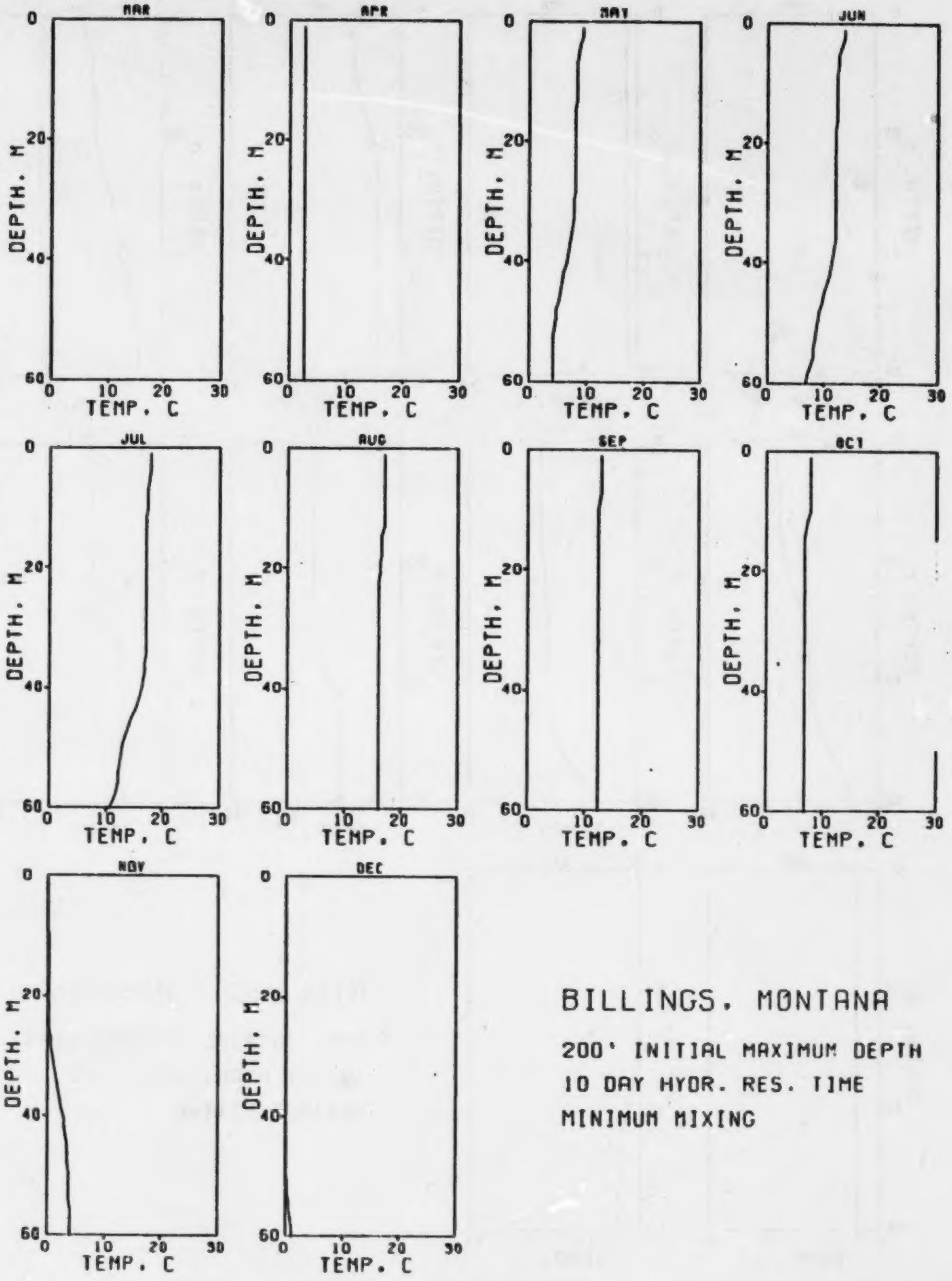
BILLINGS, MONTANA

100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



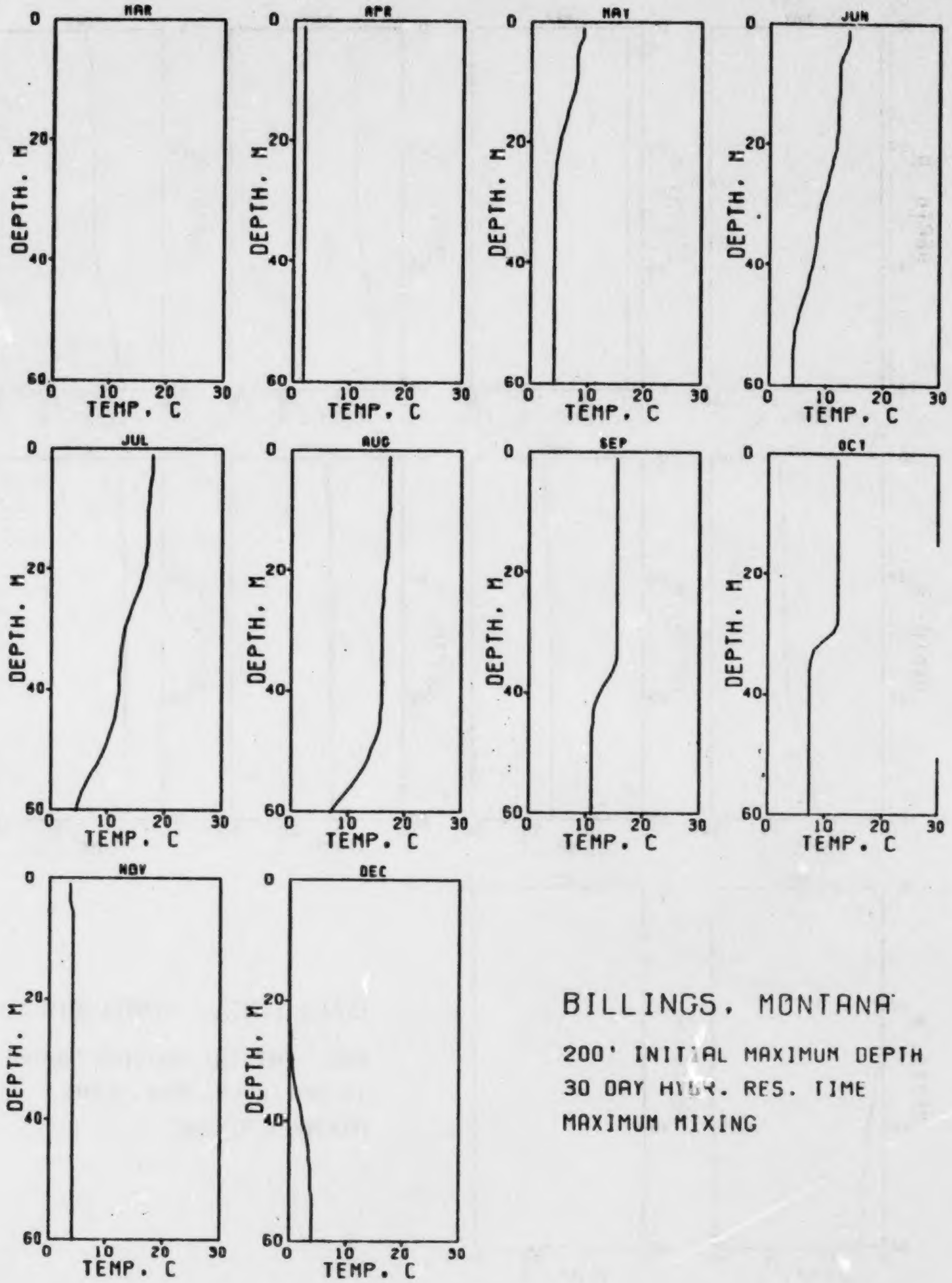
BILLINGS, MONTANA

200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



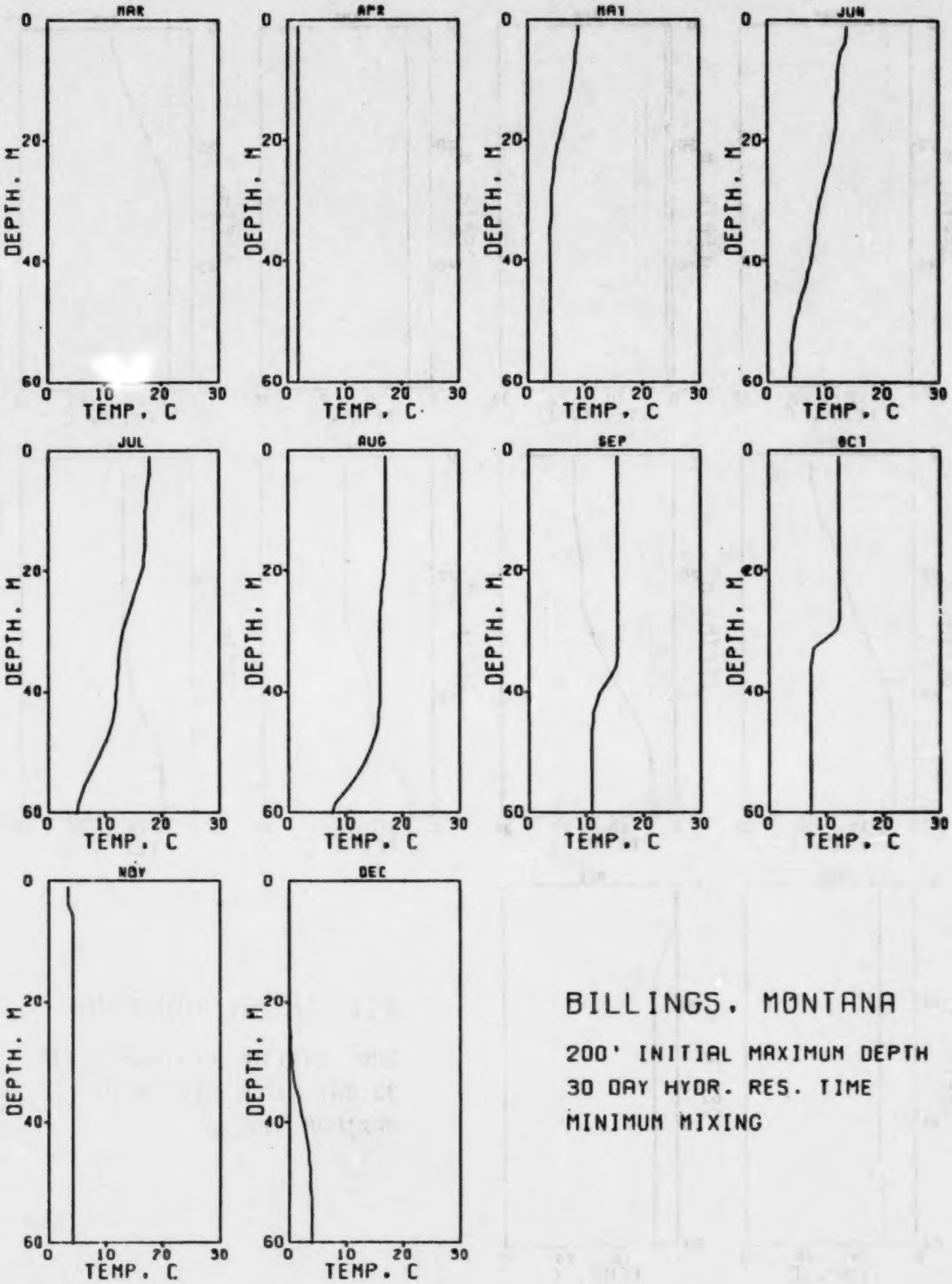
BILLINGS, MONTANA

200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



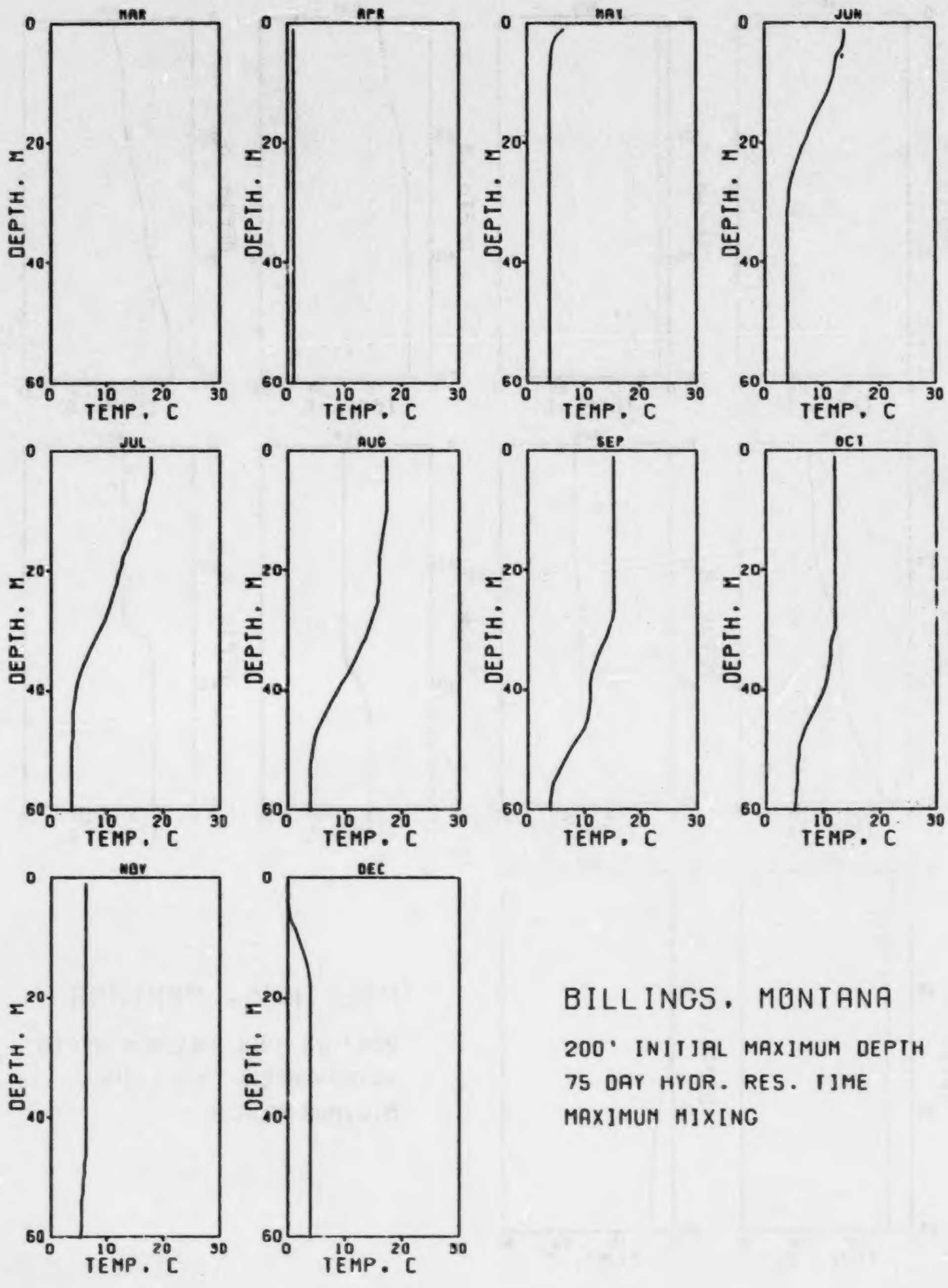
BILLINGS, MONTANA

200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



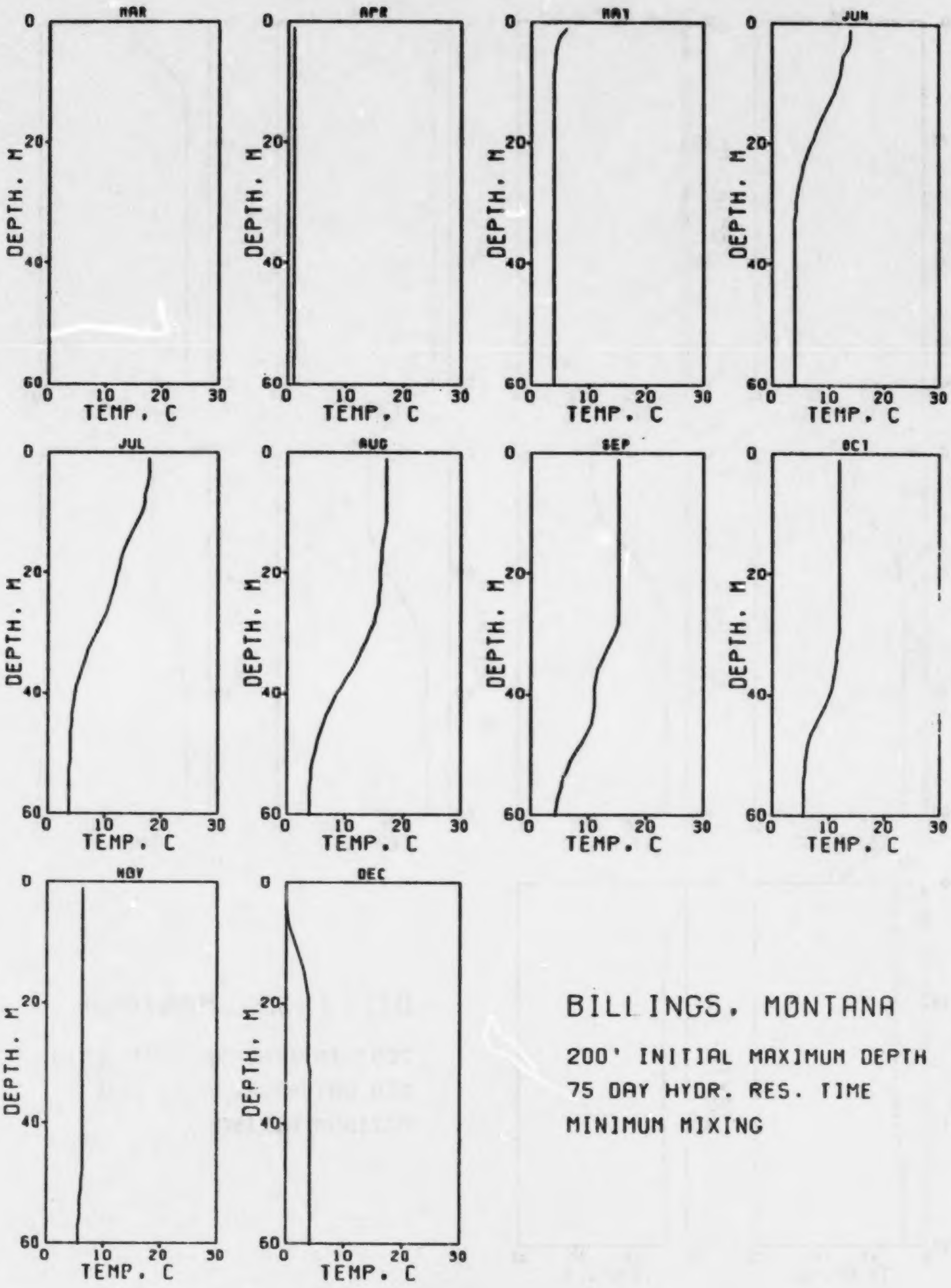
BILLINGS, MONTANA

200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



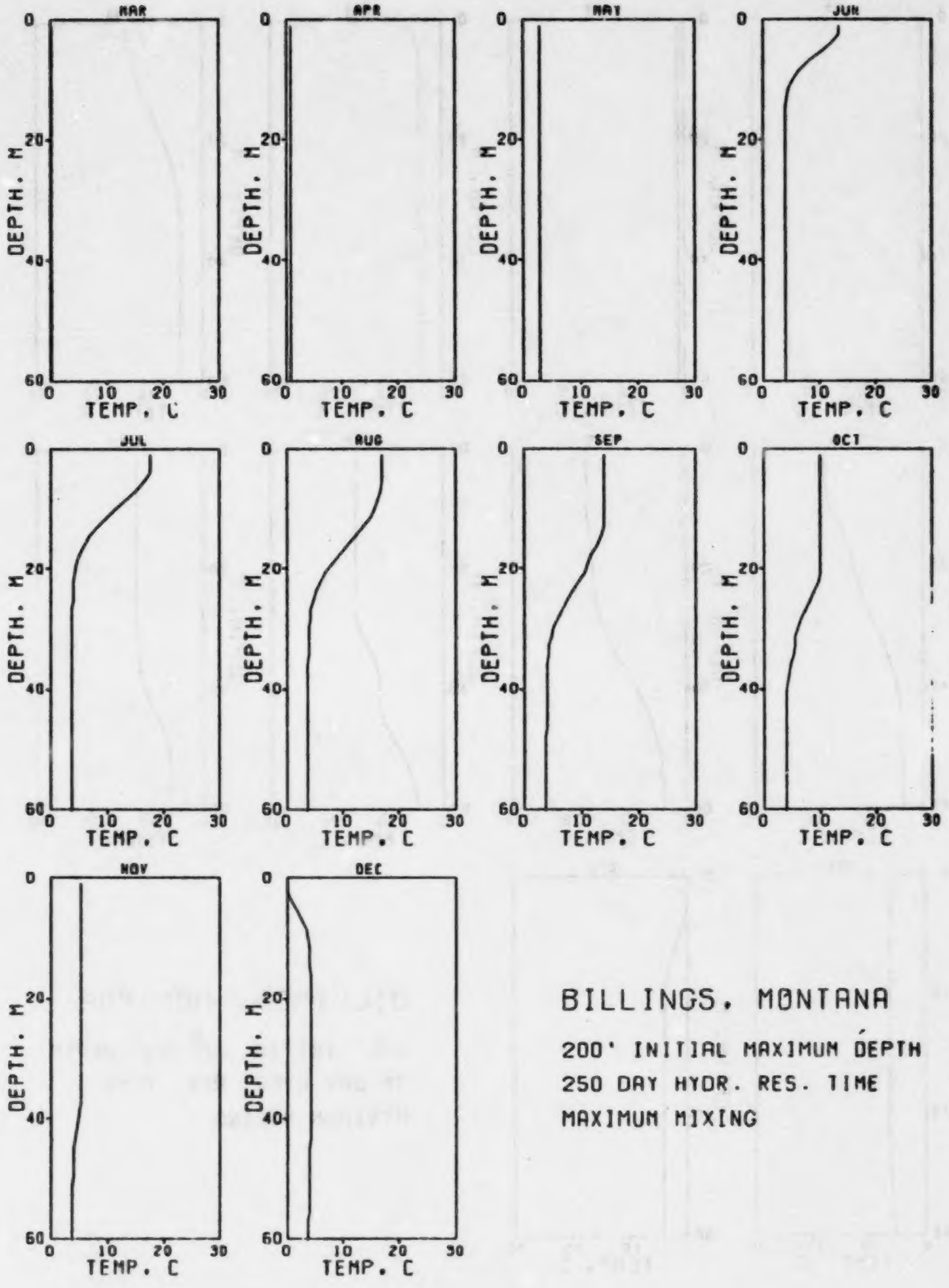
BILLINGS, MONTANA

200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



BILLINGS, MONTANA

200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING

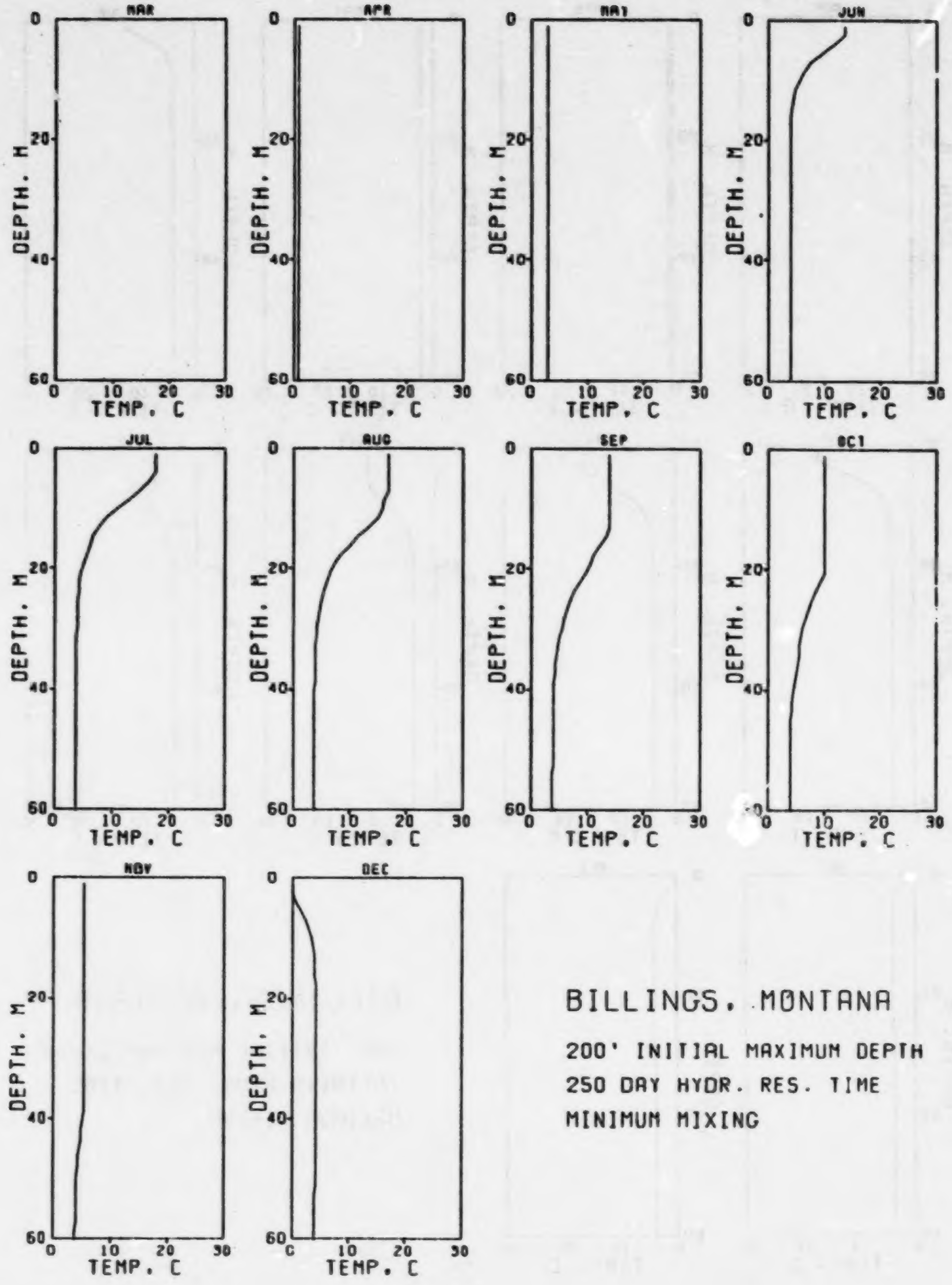


BILLINGS, MONTANA

200' INITIAL MAXIMUM DEPTH

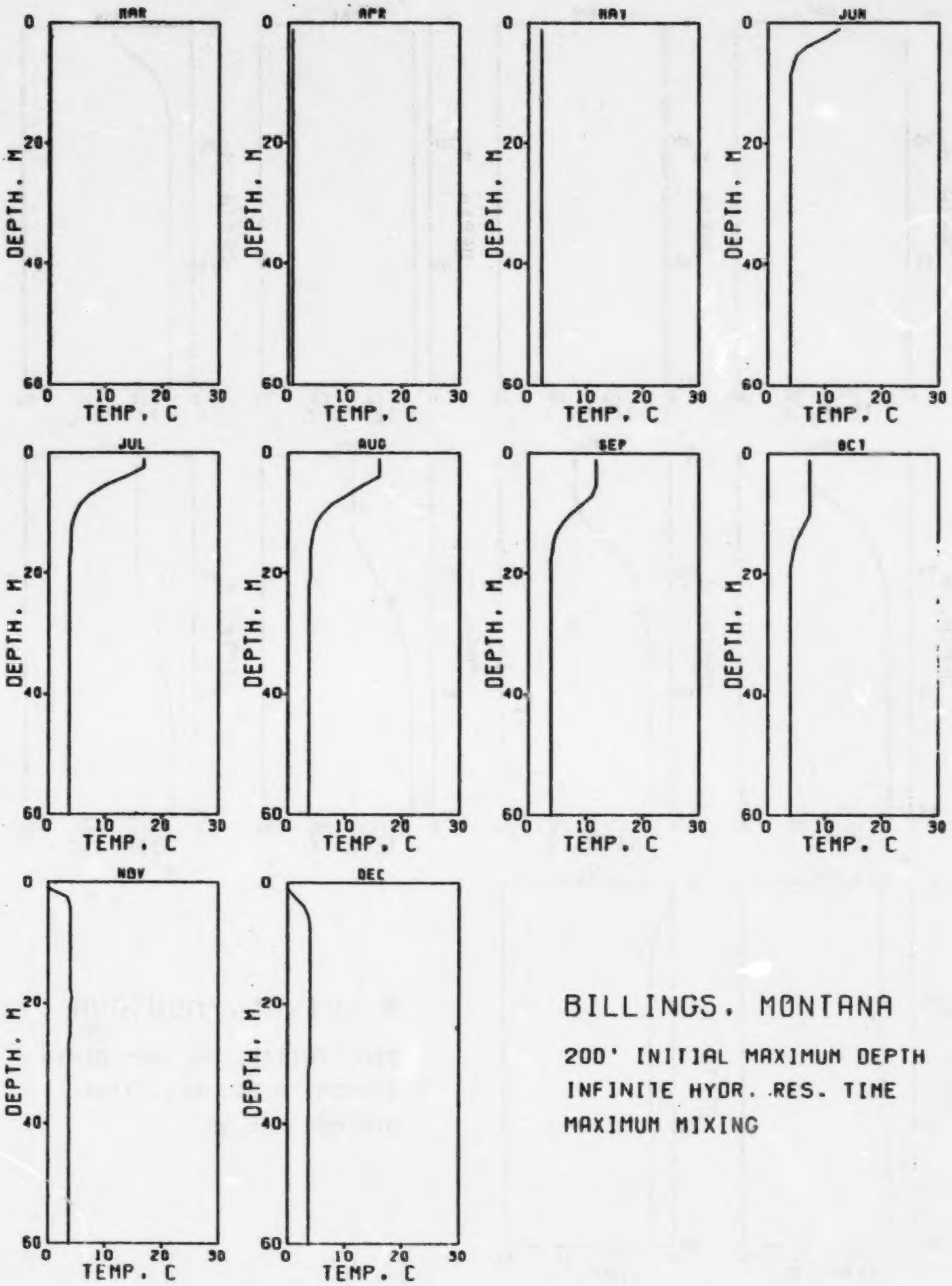
250 DAY HYDR. RES. TIME

MAXIMUM MIXING



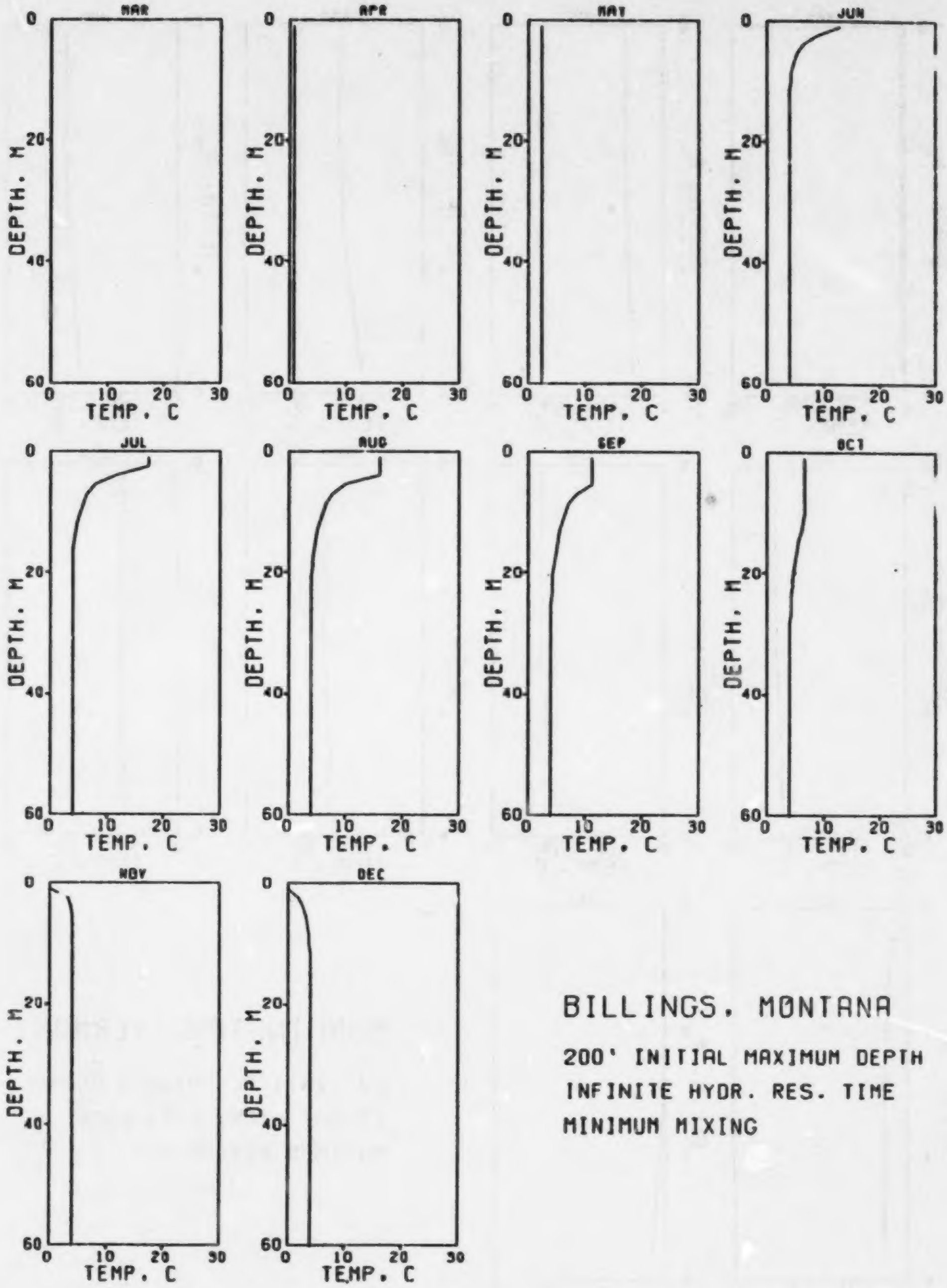
BILLINGS, MONTANA

200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



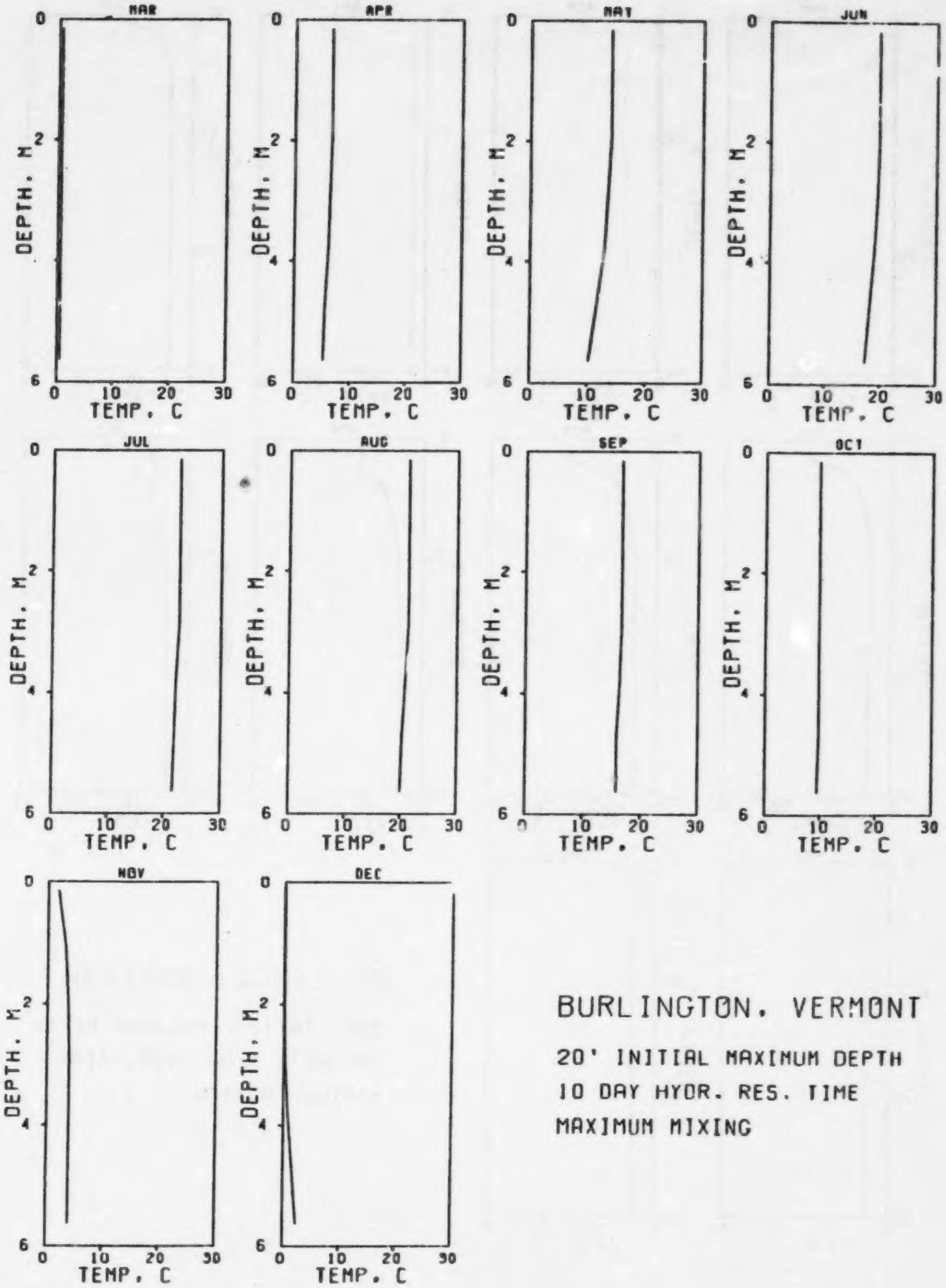
BILLINGS, MONTANA

200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



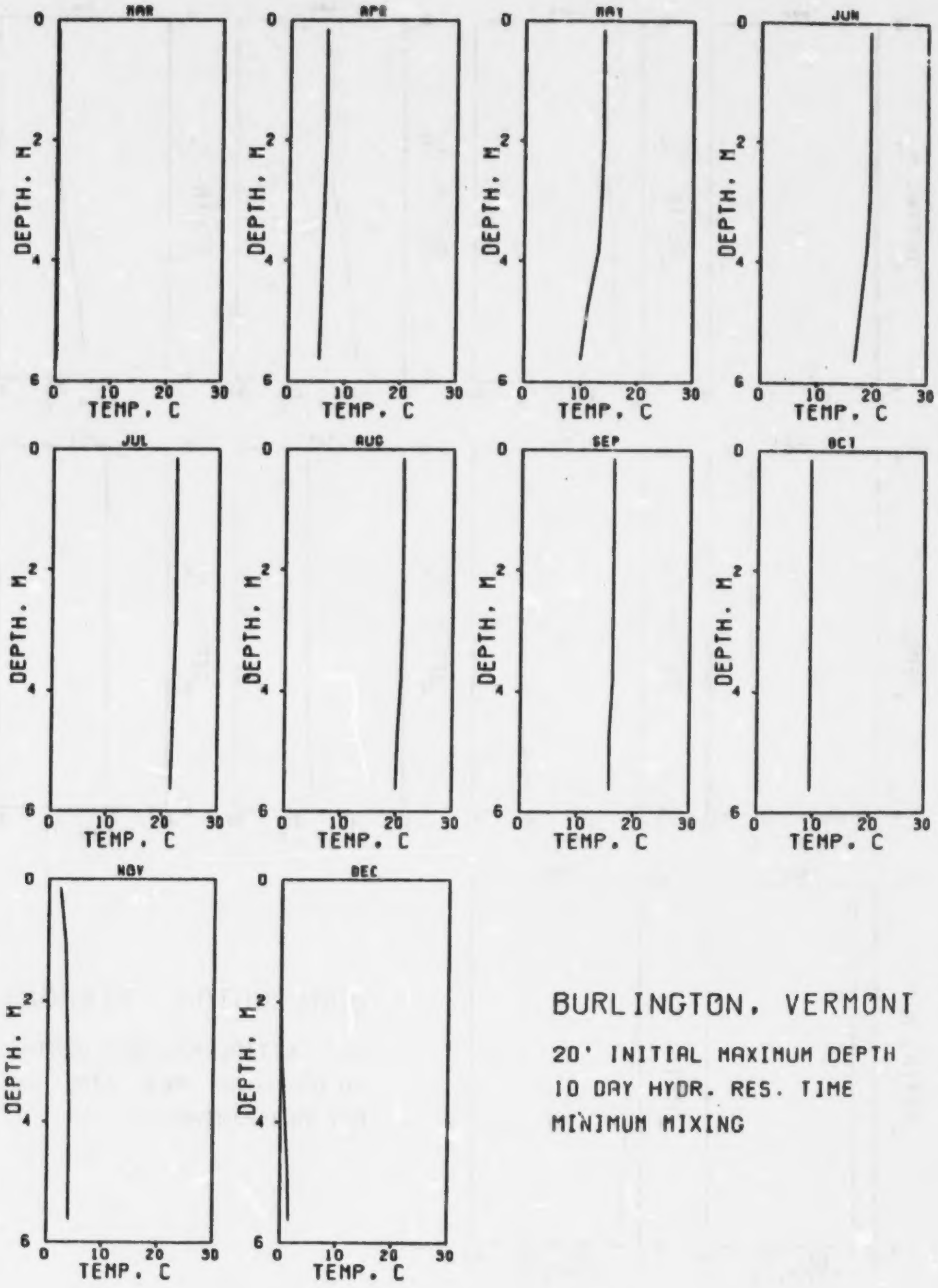
BILLINGS, MONTANA

200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



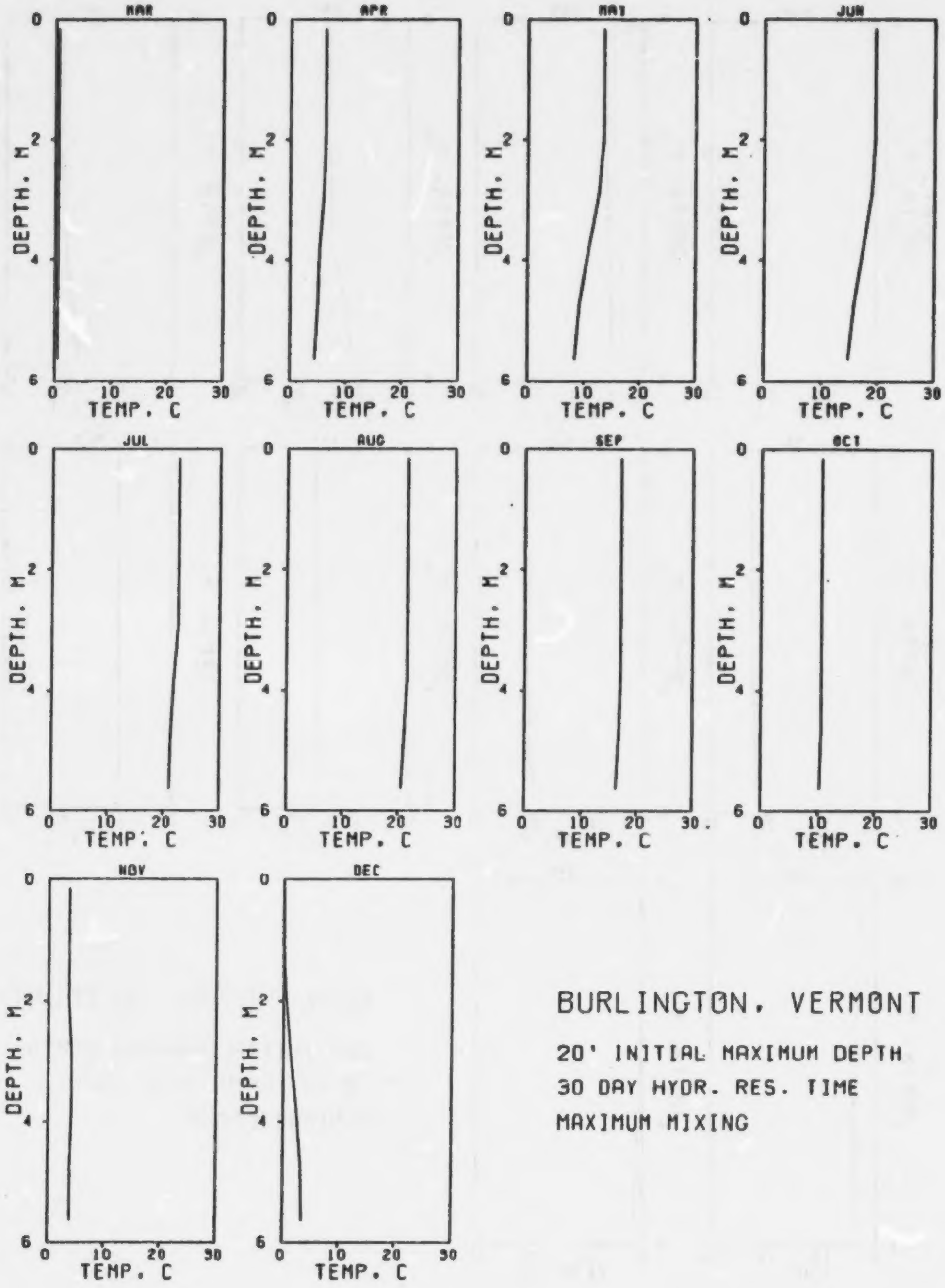
BURLINGTON, VERMONT

20' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



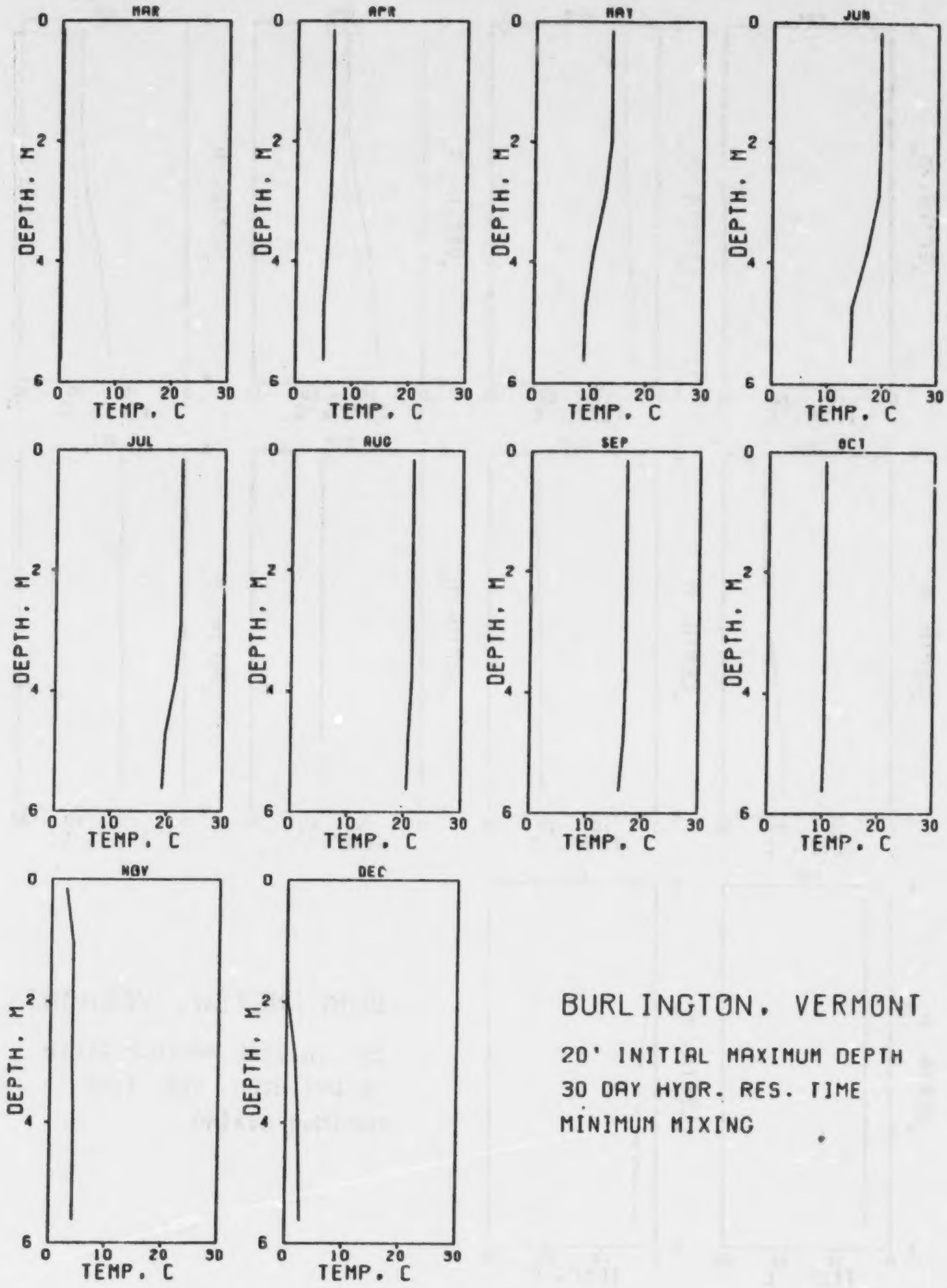
BURLINGTON, VERMONT

20' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



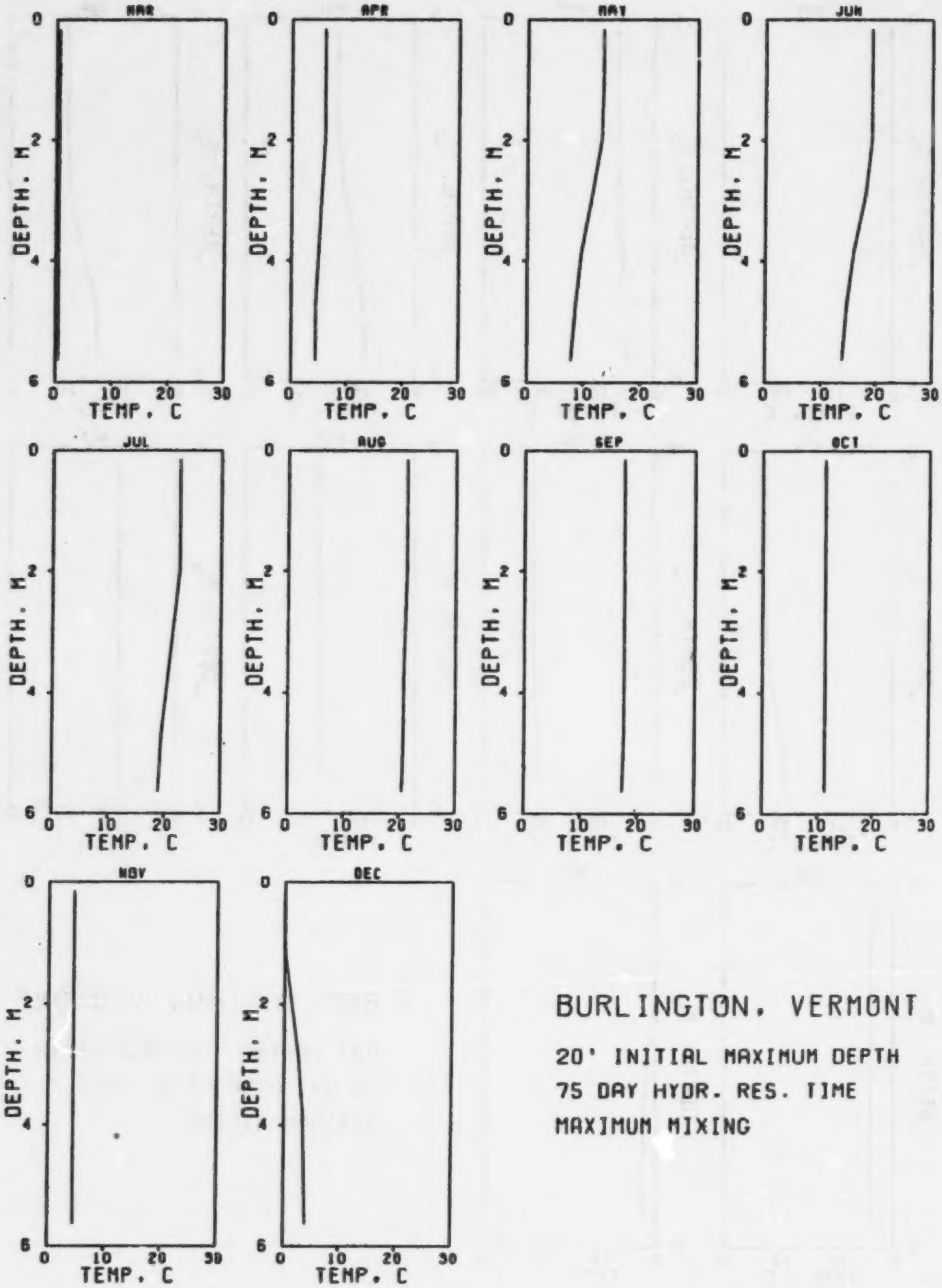
BURLINGTON, VERMONT

20' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



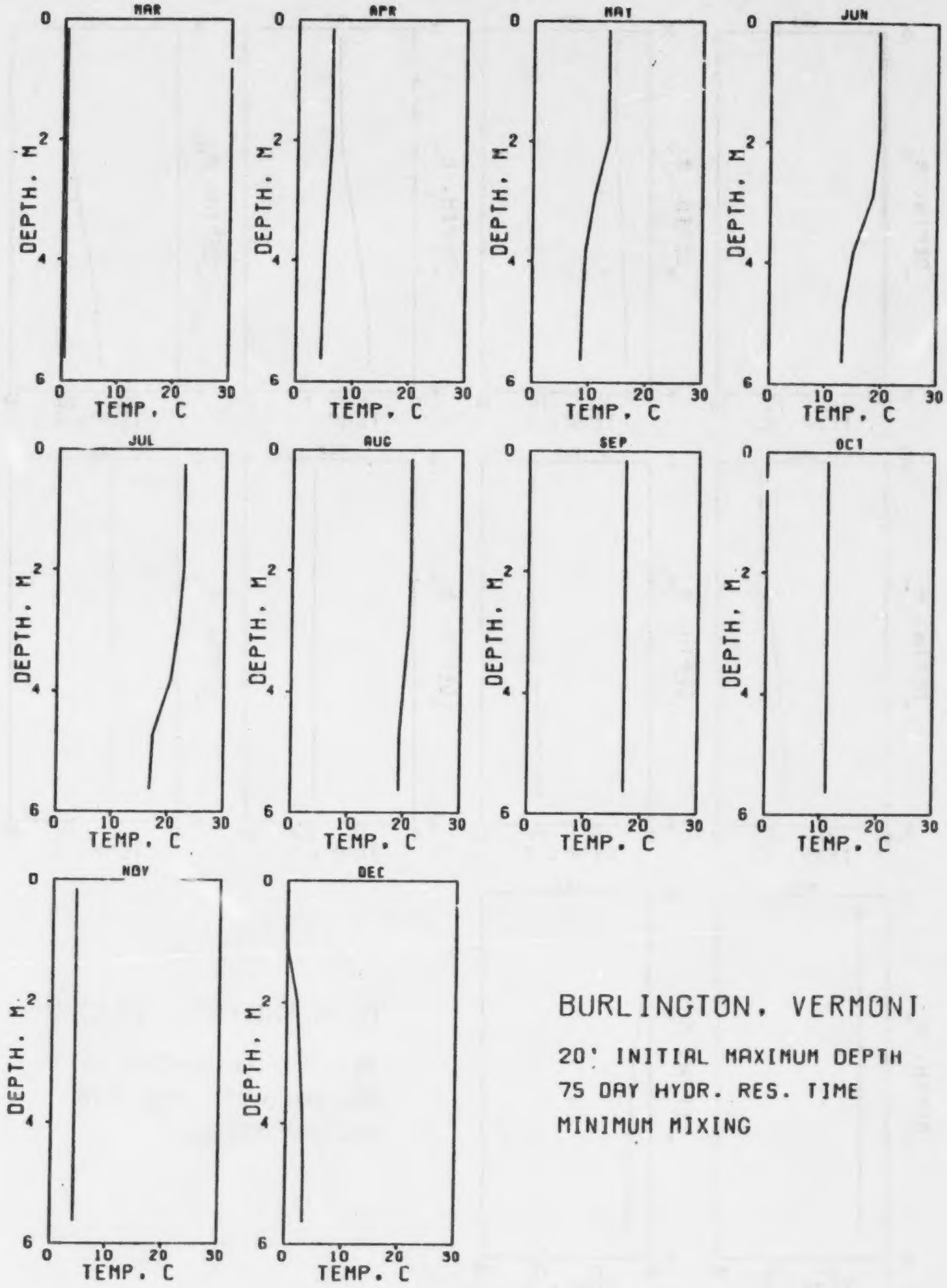
BURLINGTON, VERMONT

20' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



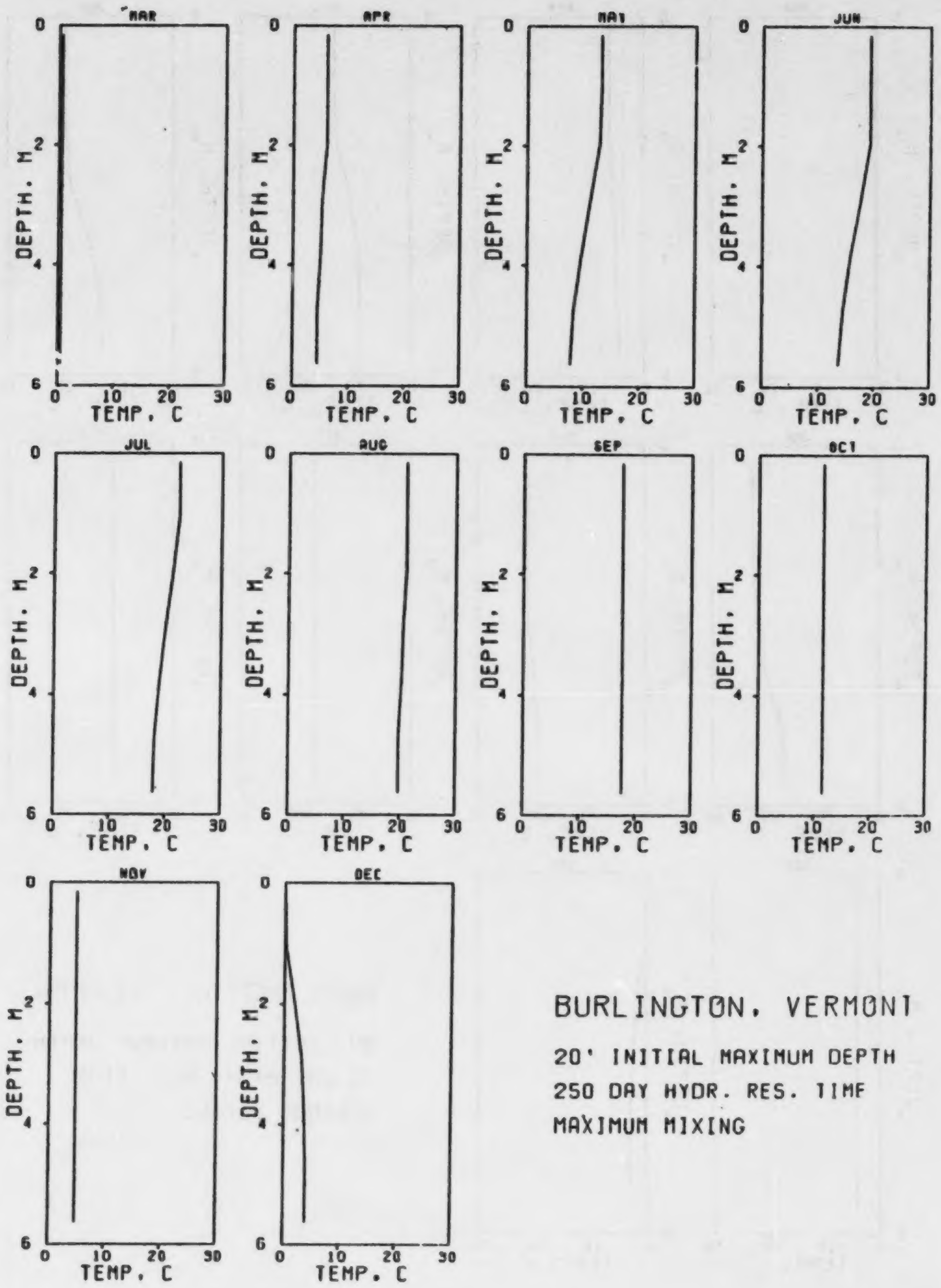
BURLINGTON, VERMONT

20' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



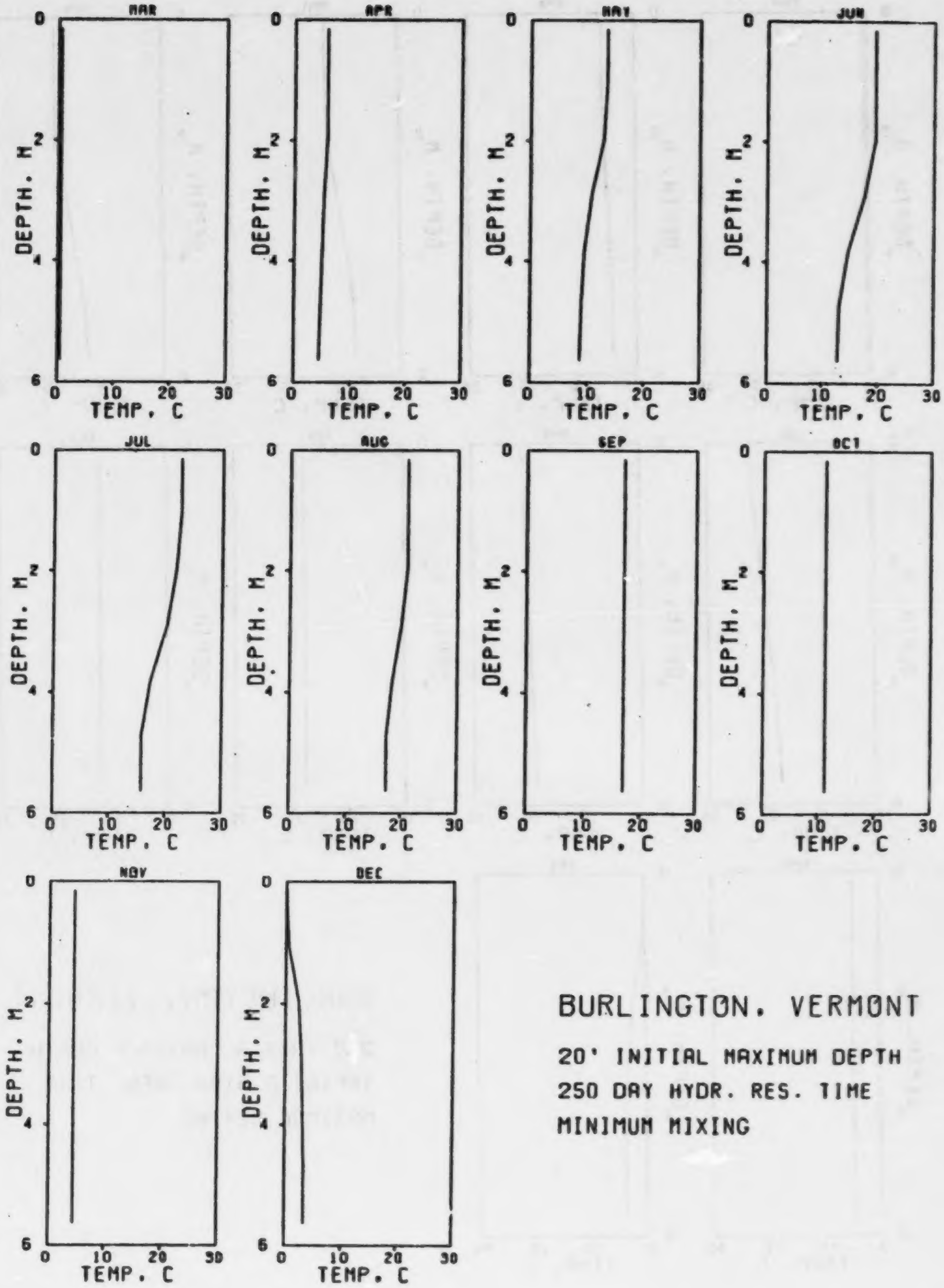
BURLINGTON, VERMONT

20' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



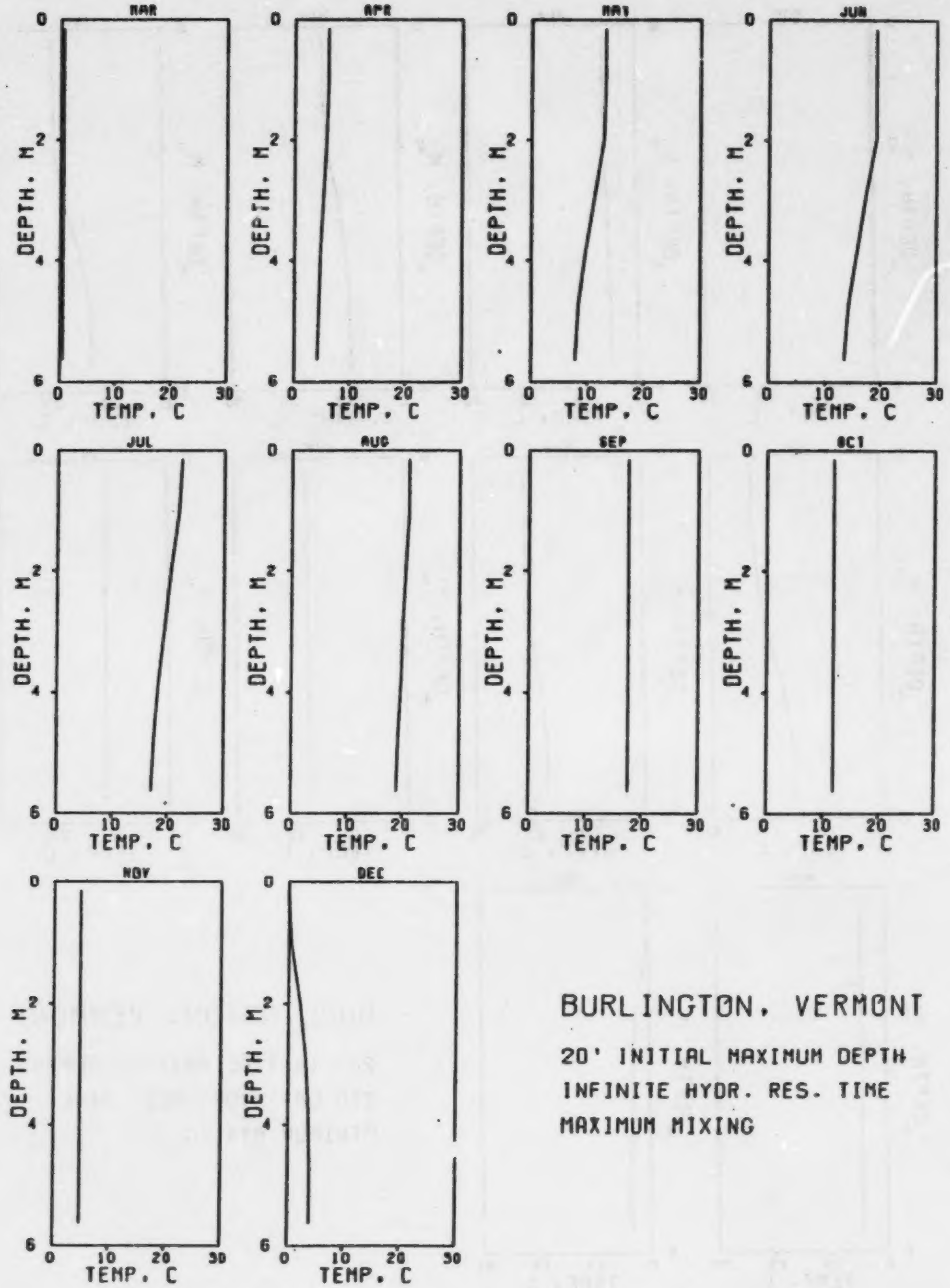
BURLINGTON, VERMONT

20' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



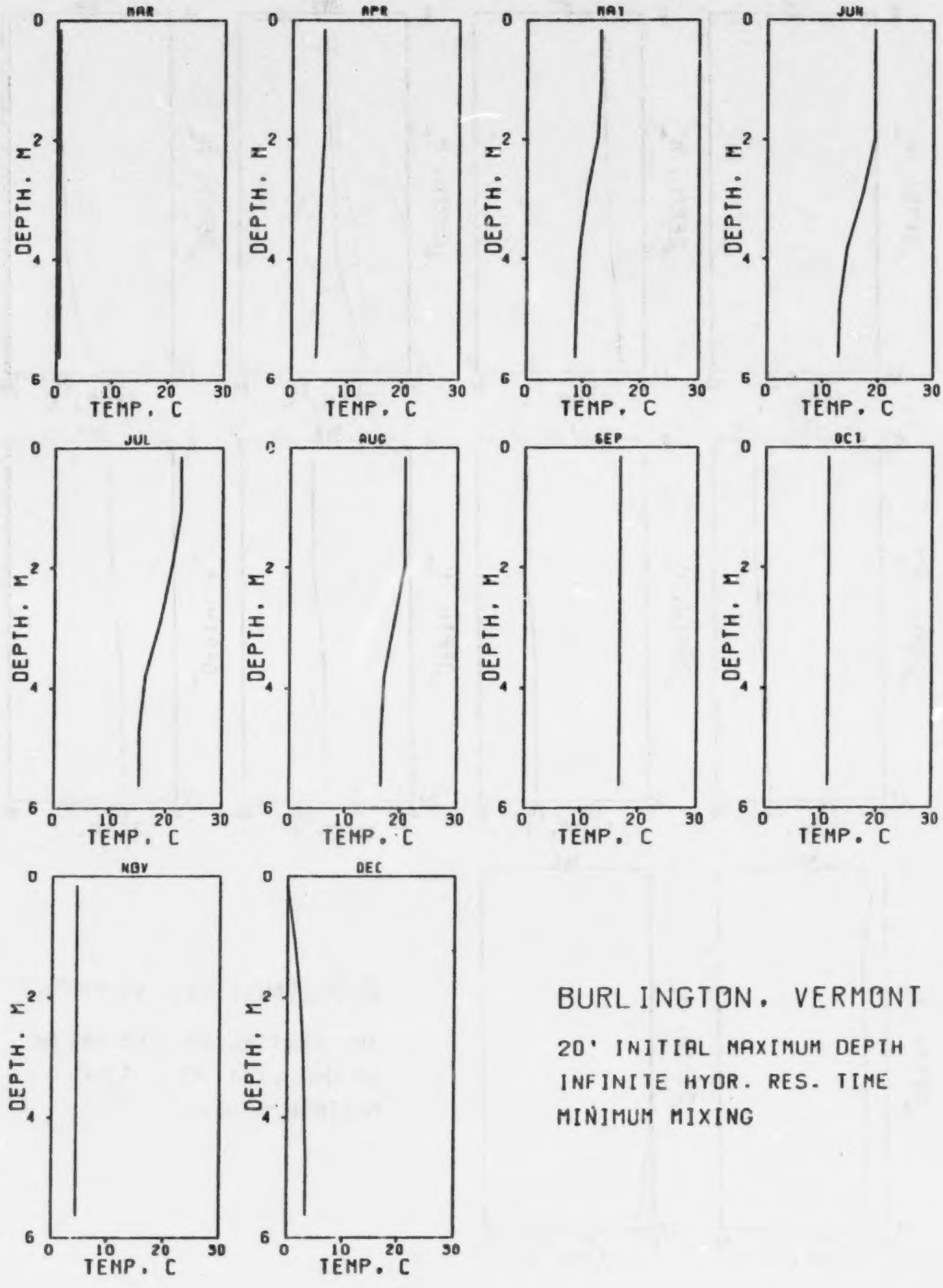
BURLINGTON, VERMONT

20' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



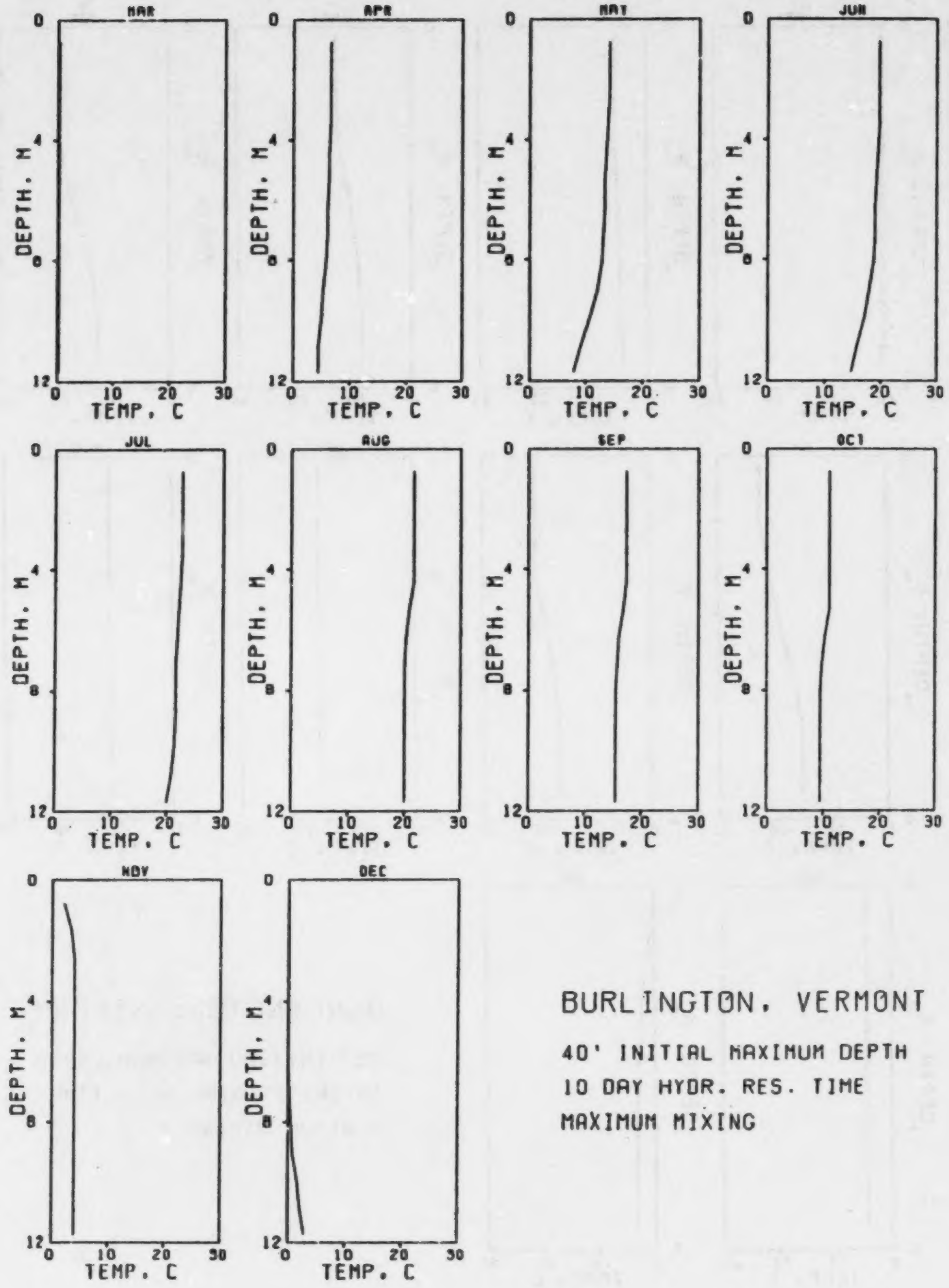
BURLINGTON, VERMONT

20' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



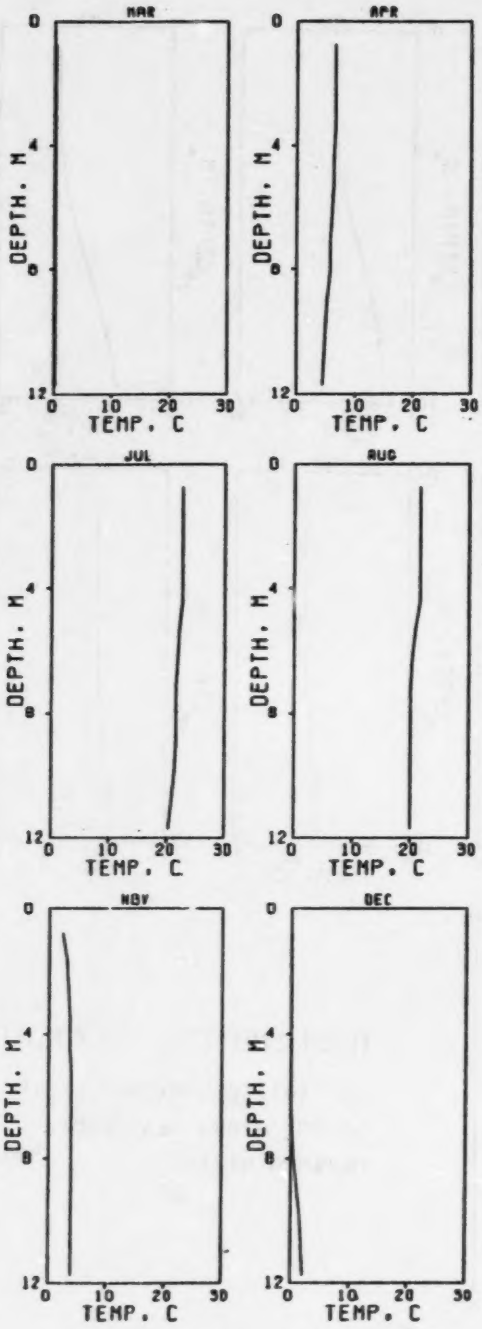
BURLINGTON, VERMONT

20' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



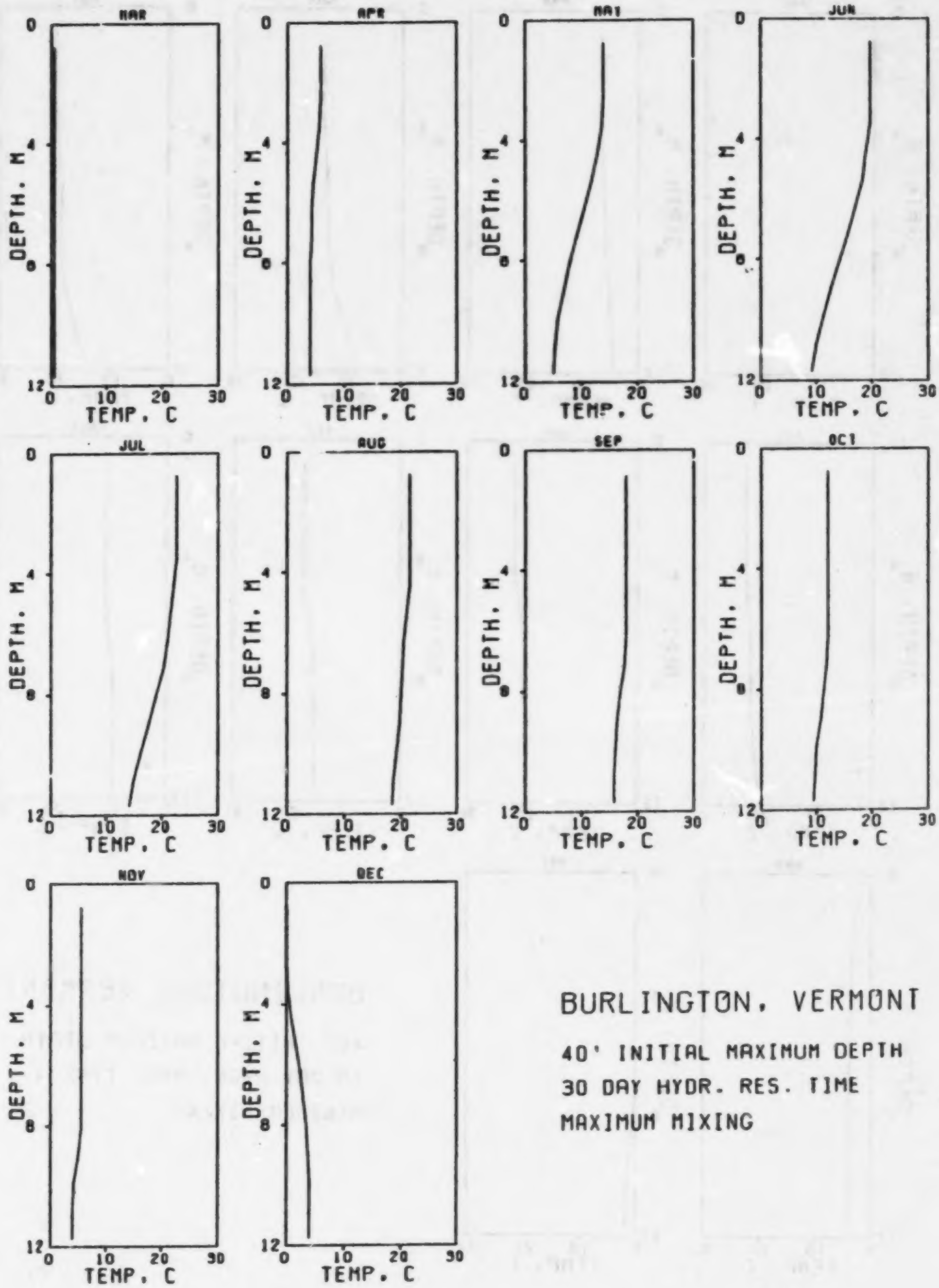
BURLINGTON, VERMONT

40' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



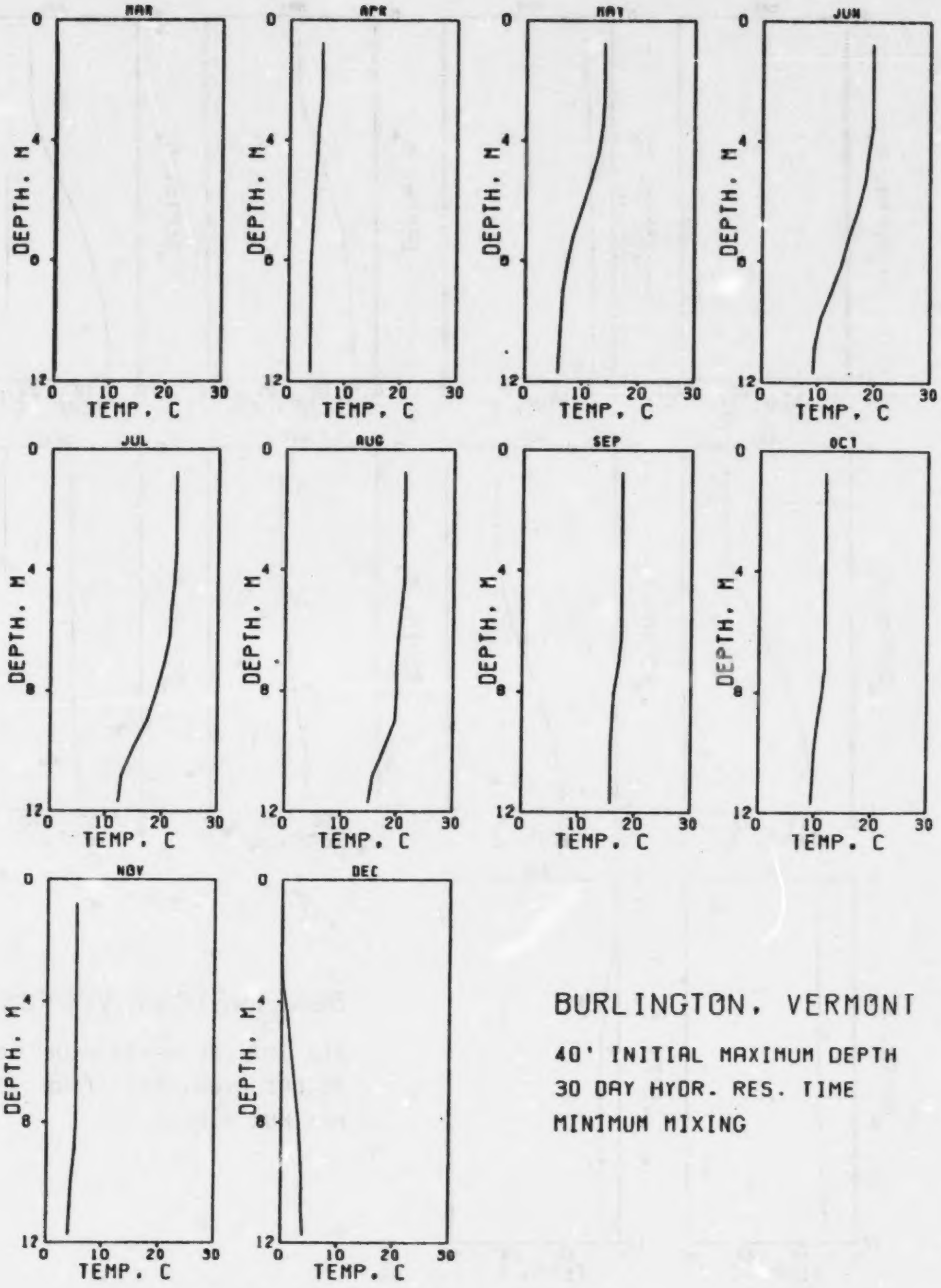
BURLINGTON, VERMONT

40' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



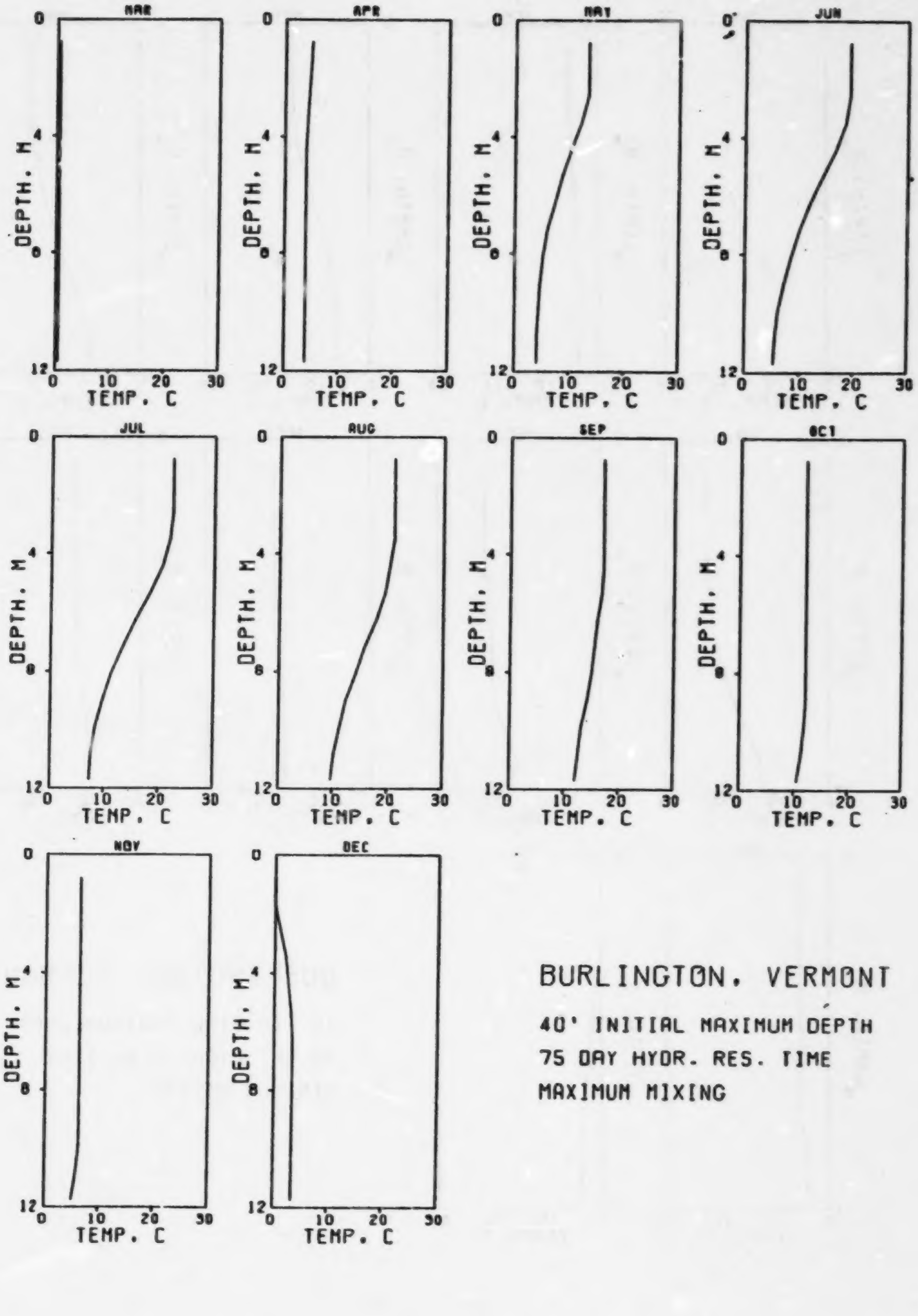
BURLINGTON, VERMONT

40' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



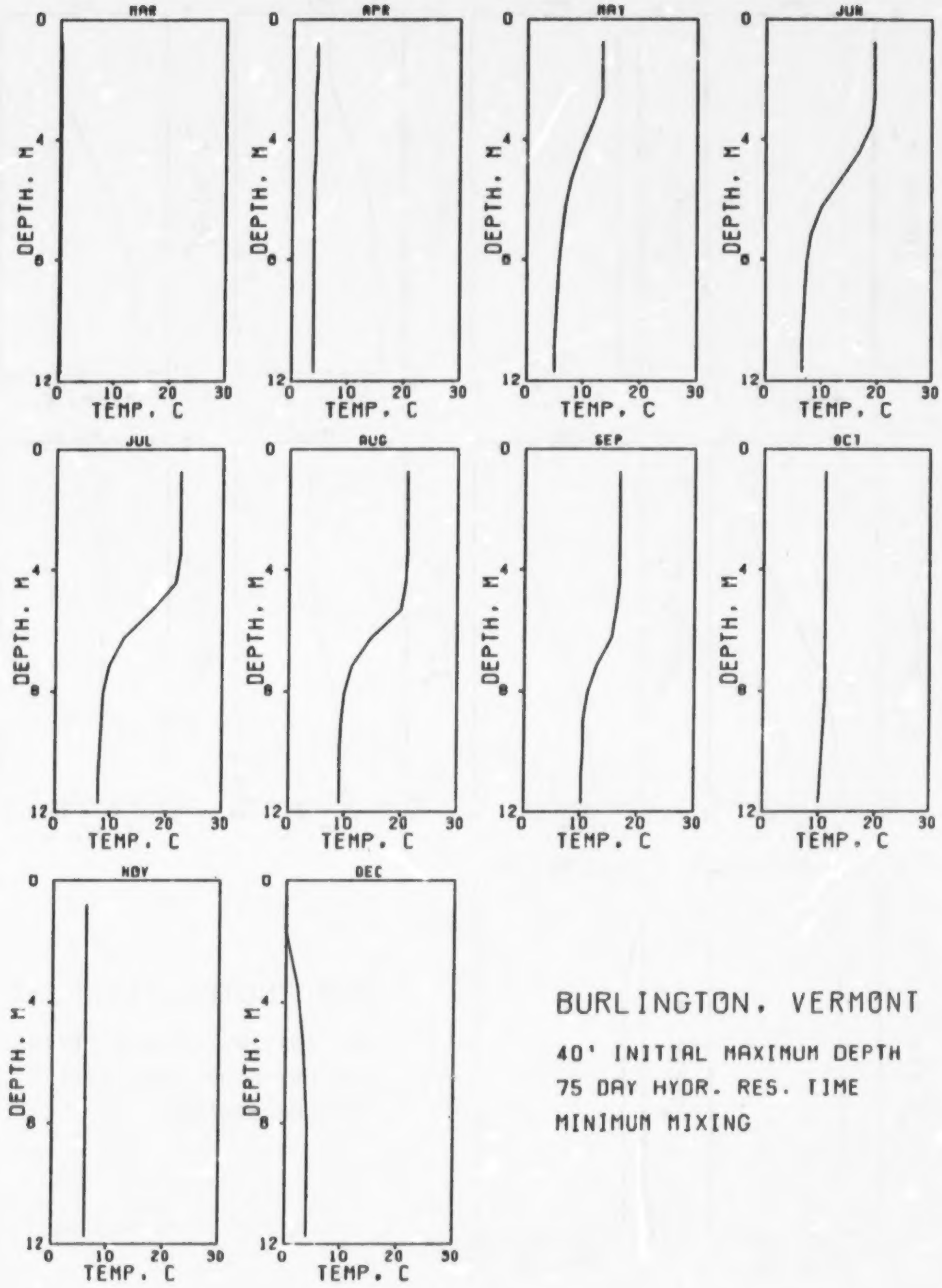
BURLINGTON, VERMONT

40' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



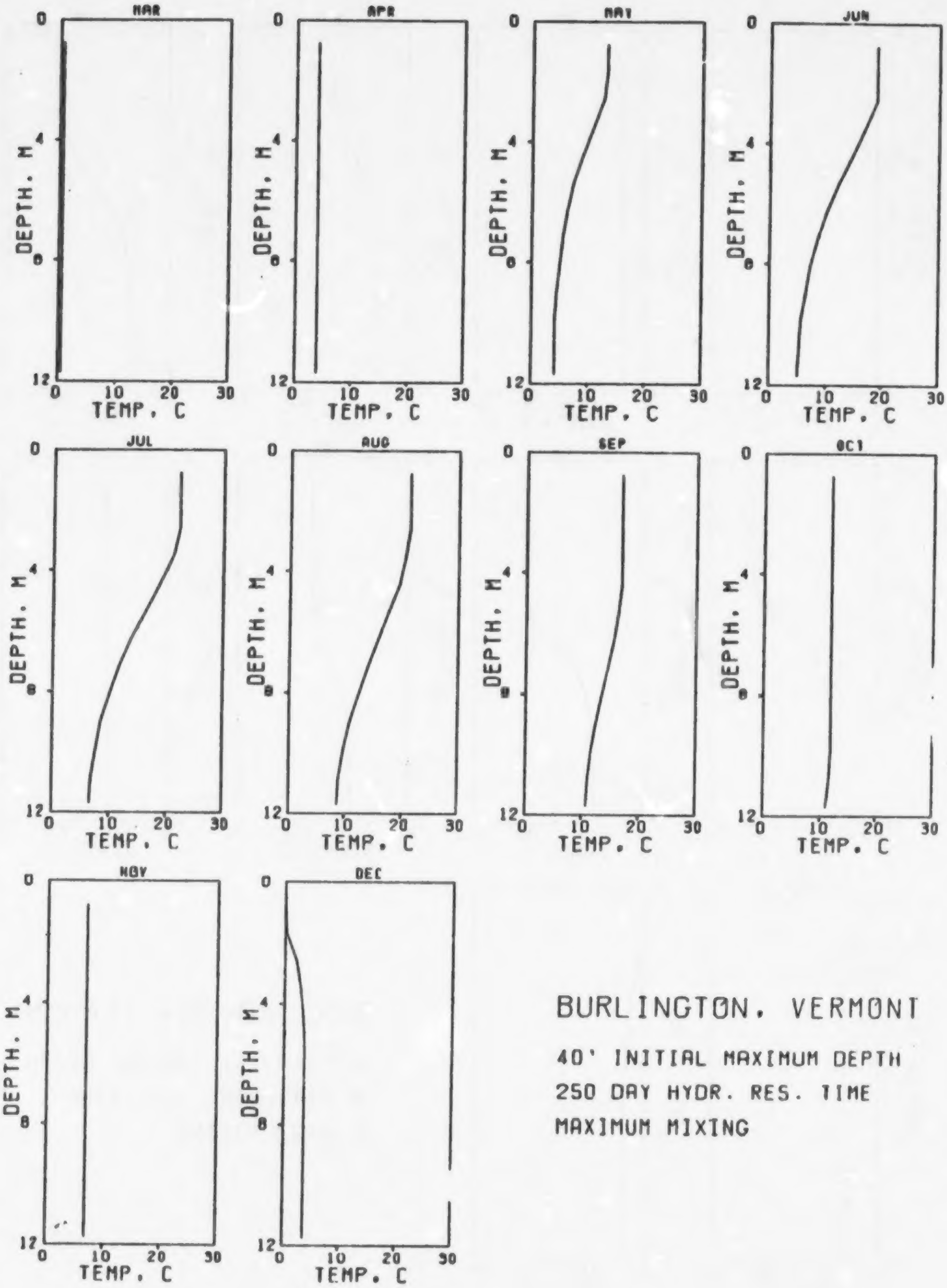
BURLINGTON, VERMONT

40' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



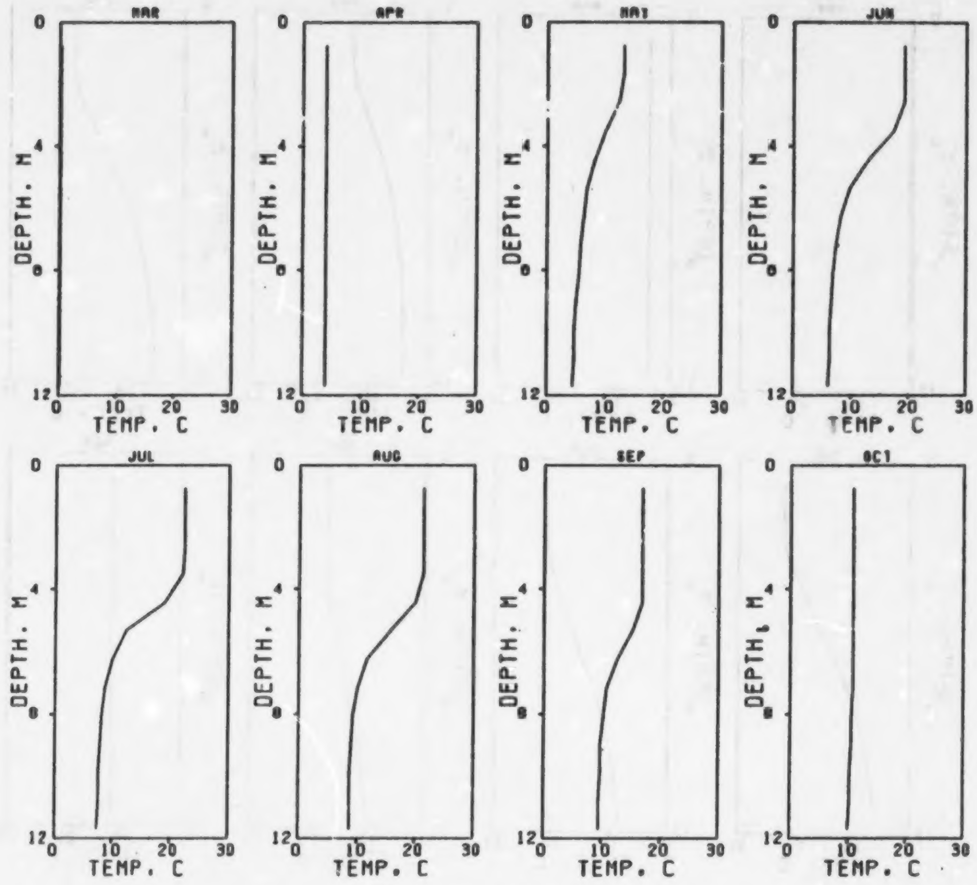
BURLINGTON, VERMONT

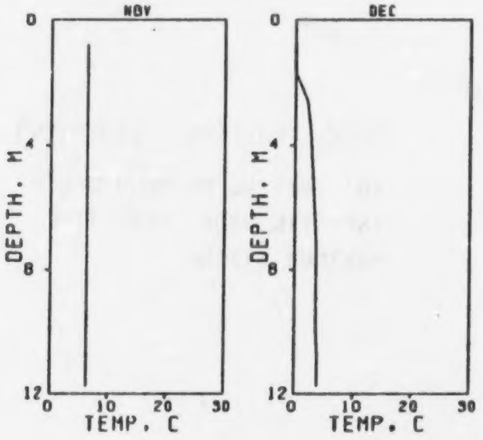
40' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



BURLINGTON, VERMONT

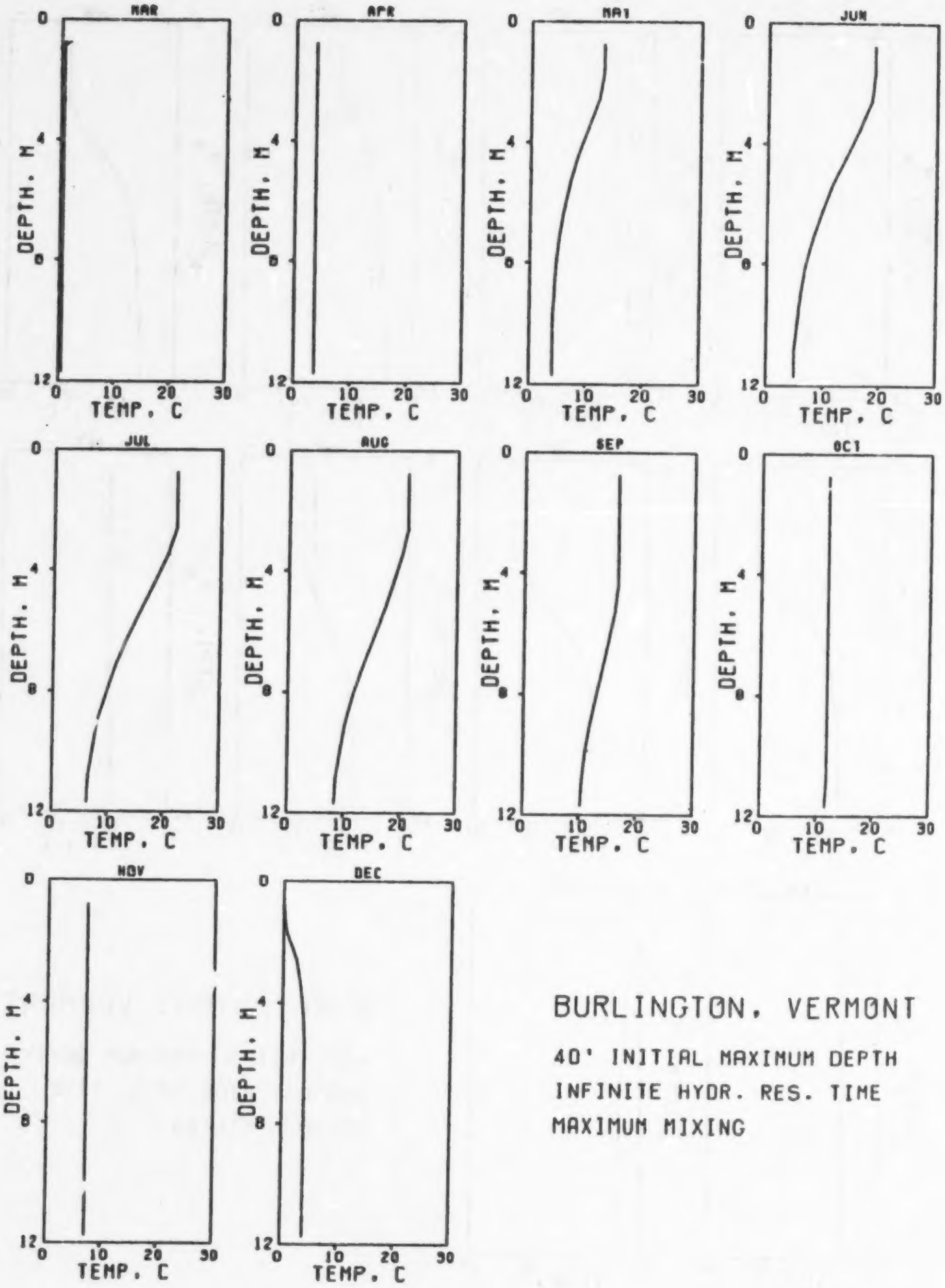
40' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING





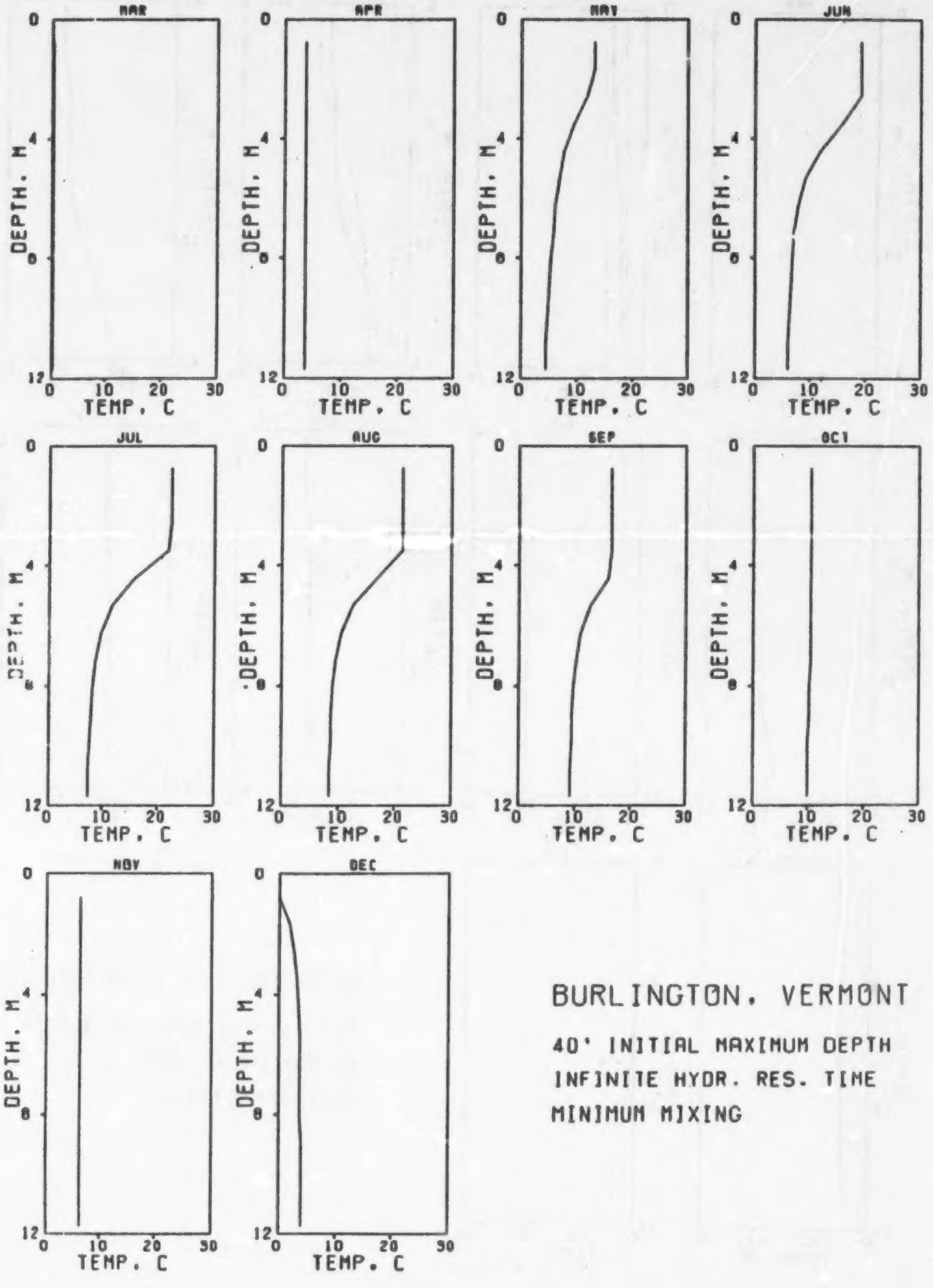
BURLINGTON, VERMONT

40' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



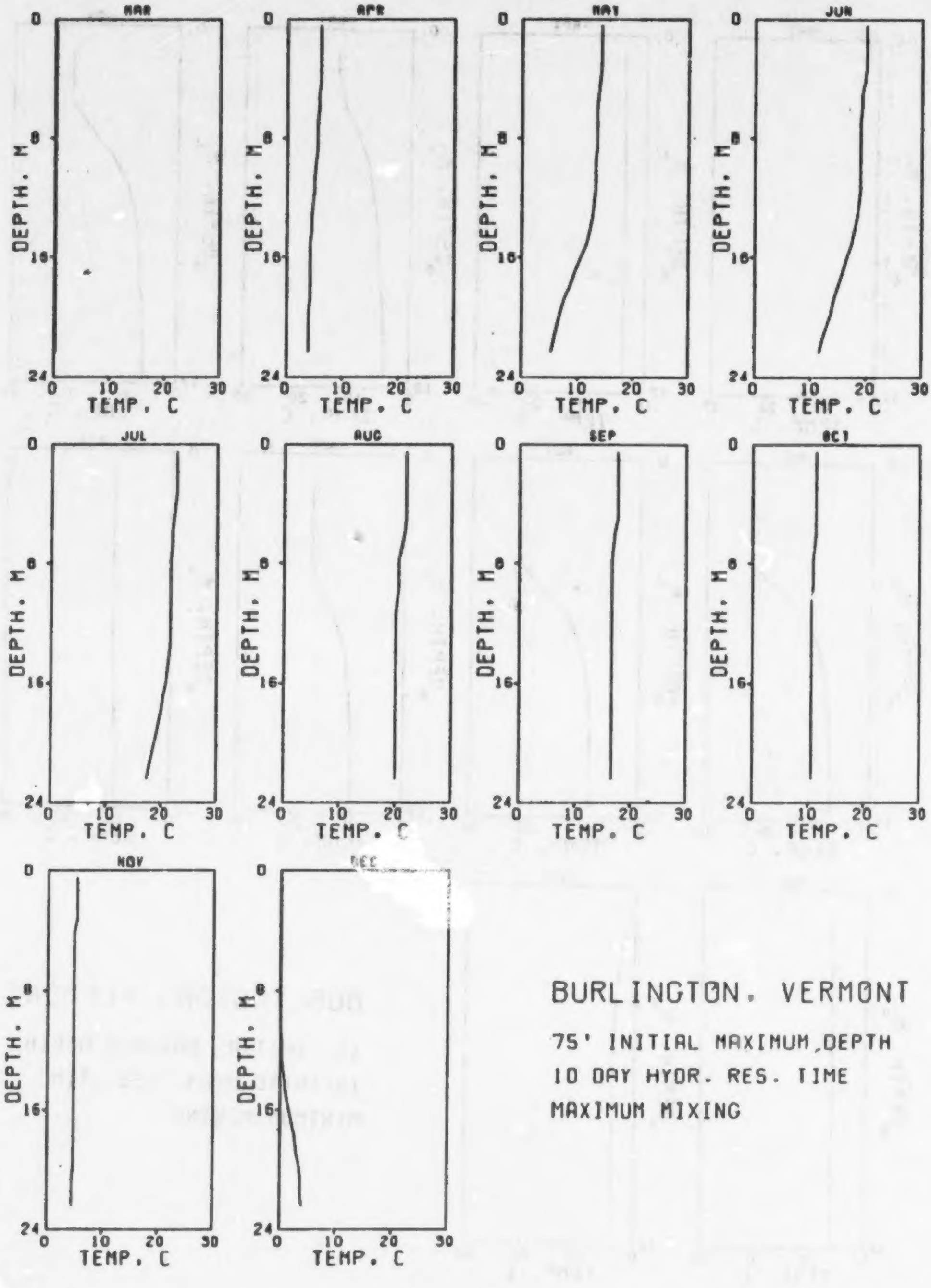
BURLINGTON, VERMONT

40' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



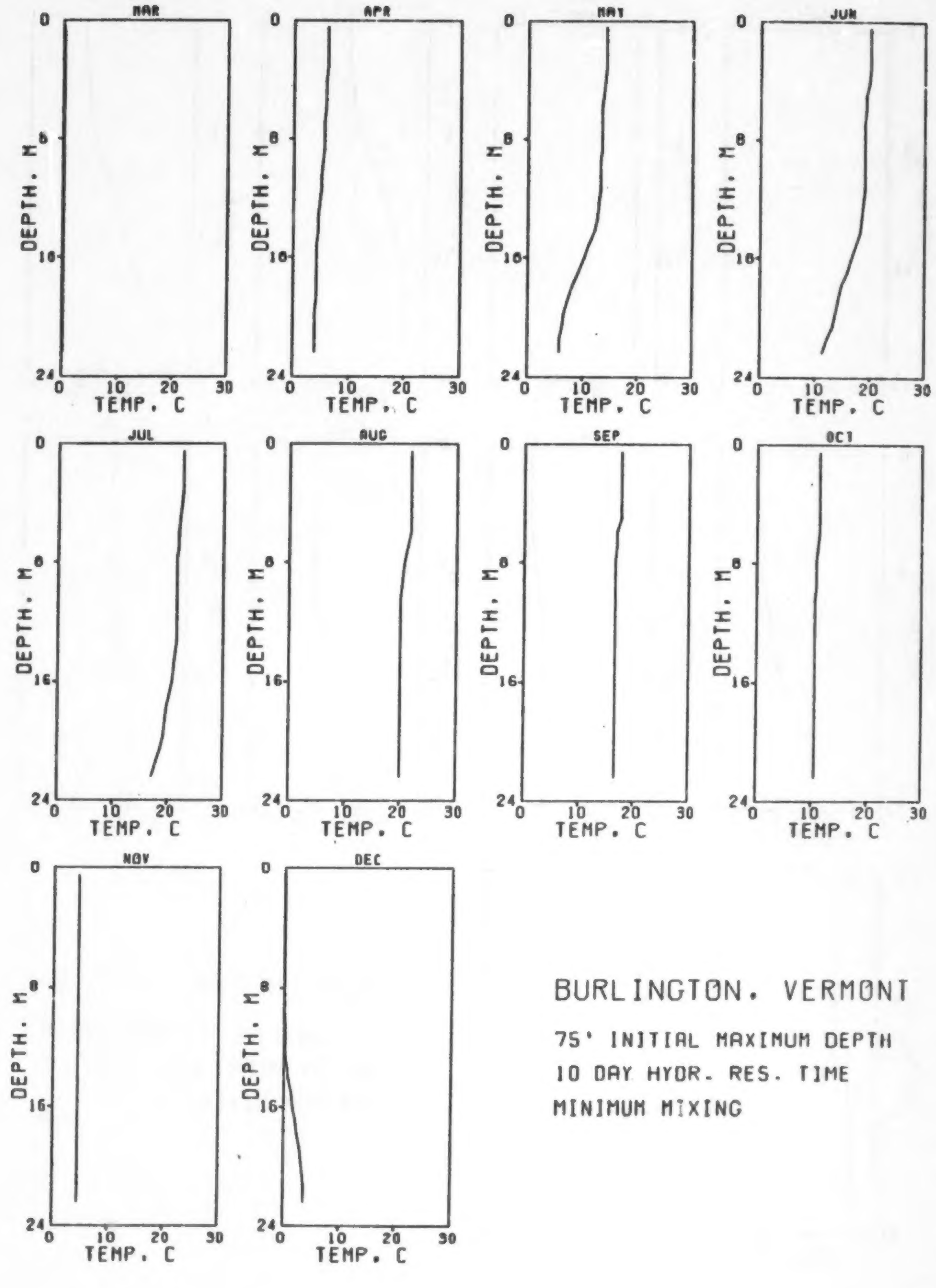
BURLINGTON, VERMONT

40' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



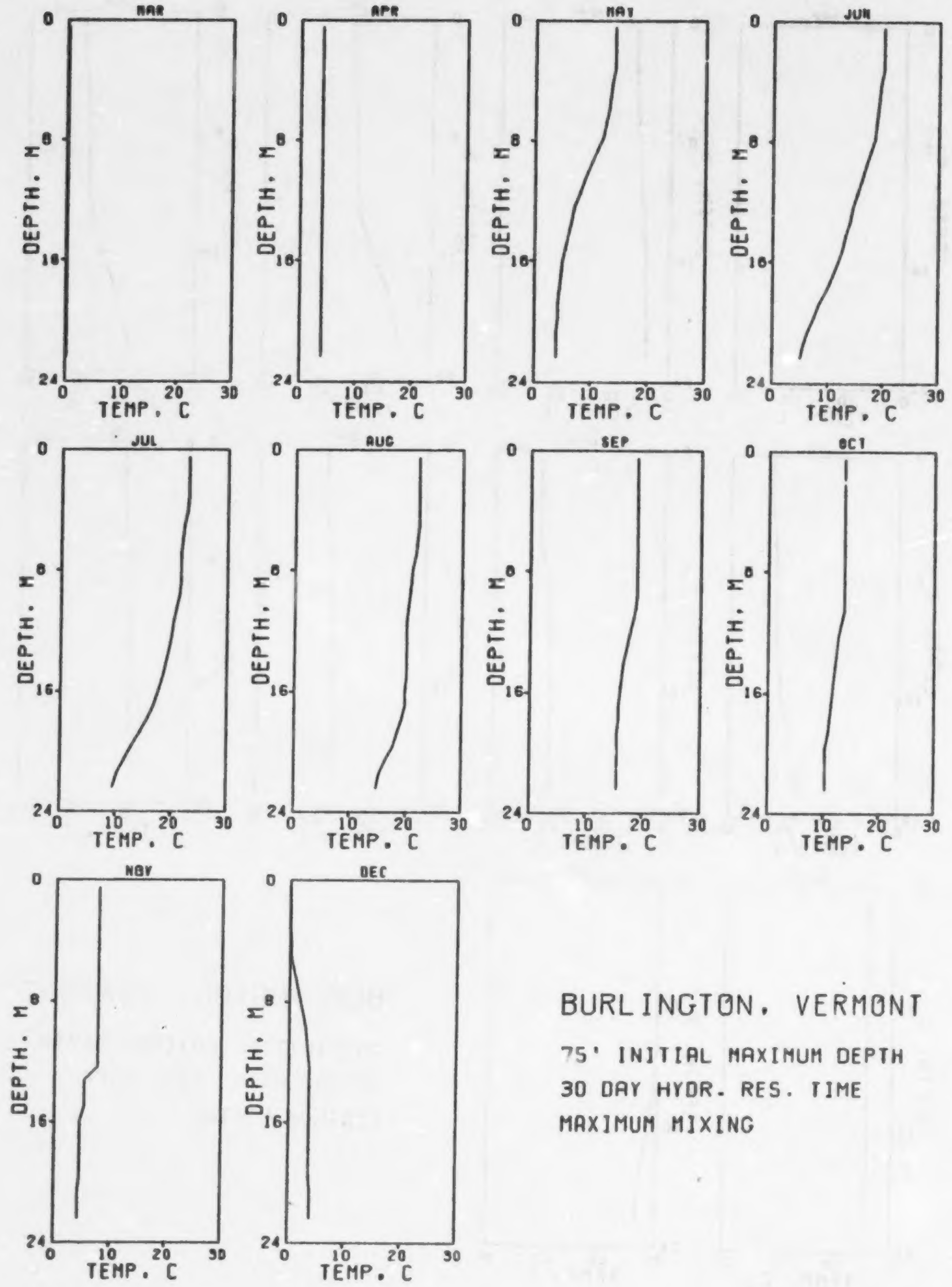
BURLINGTON, VERMONT

75' INITIAL MAXIMUM,DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



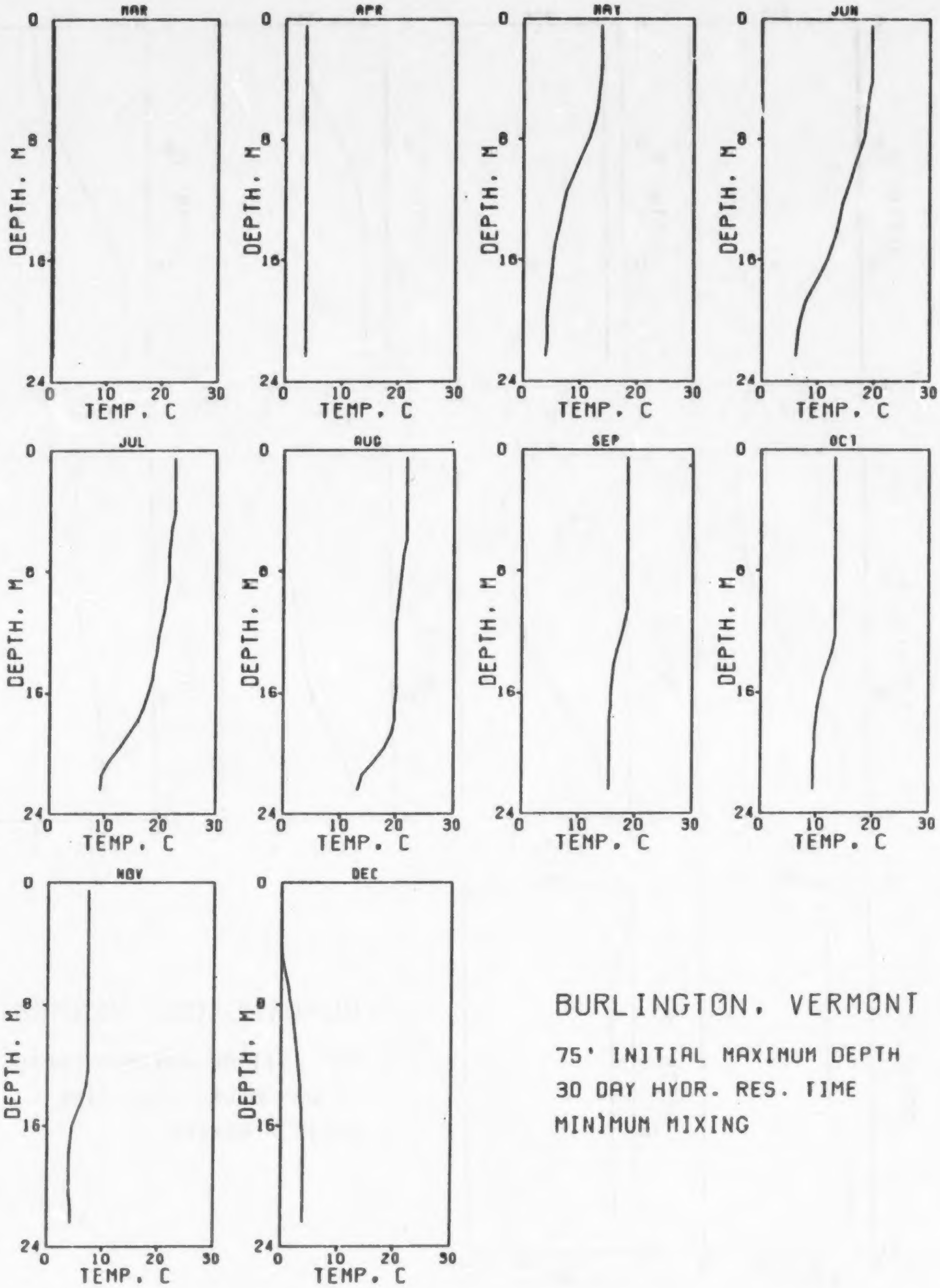
BURLINGTON, VERMONT

75' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



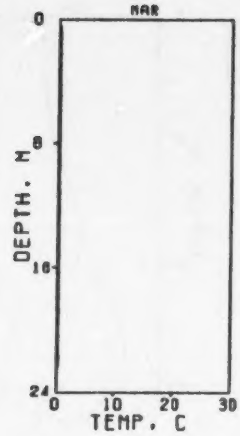
BURLINGTON, VERMONT

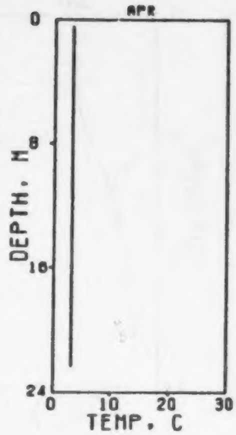
75' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING

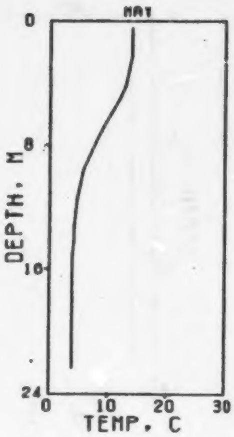


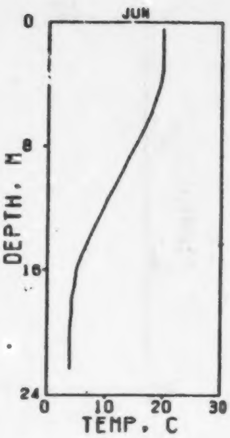
BURLINGTON, VERMONT

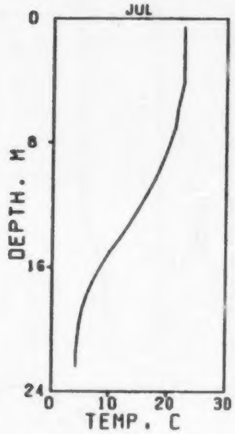
75' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING

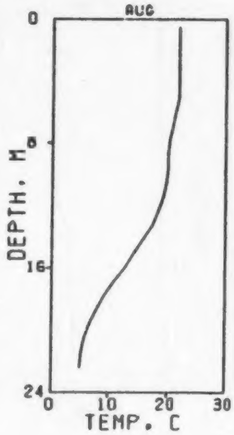


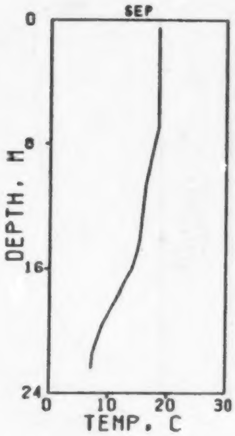


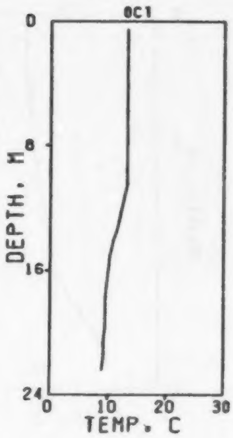


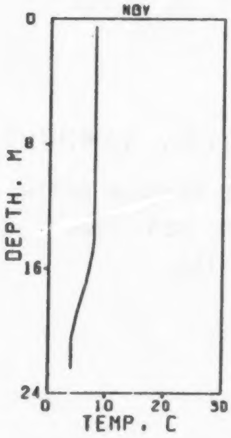


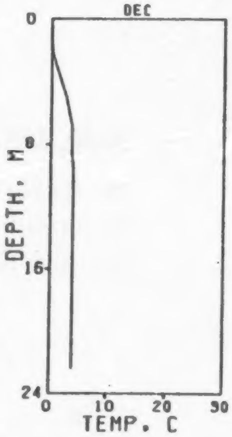






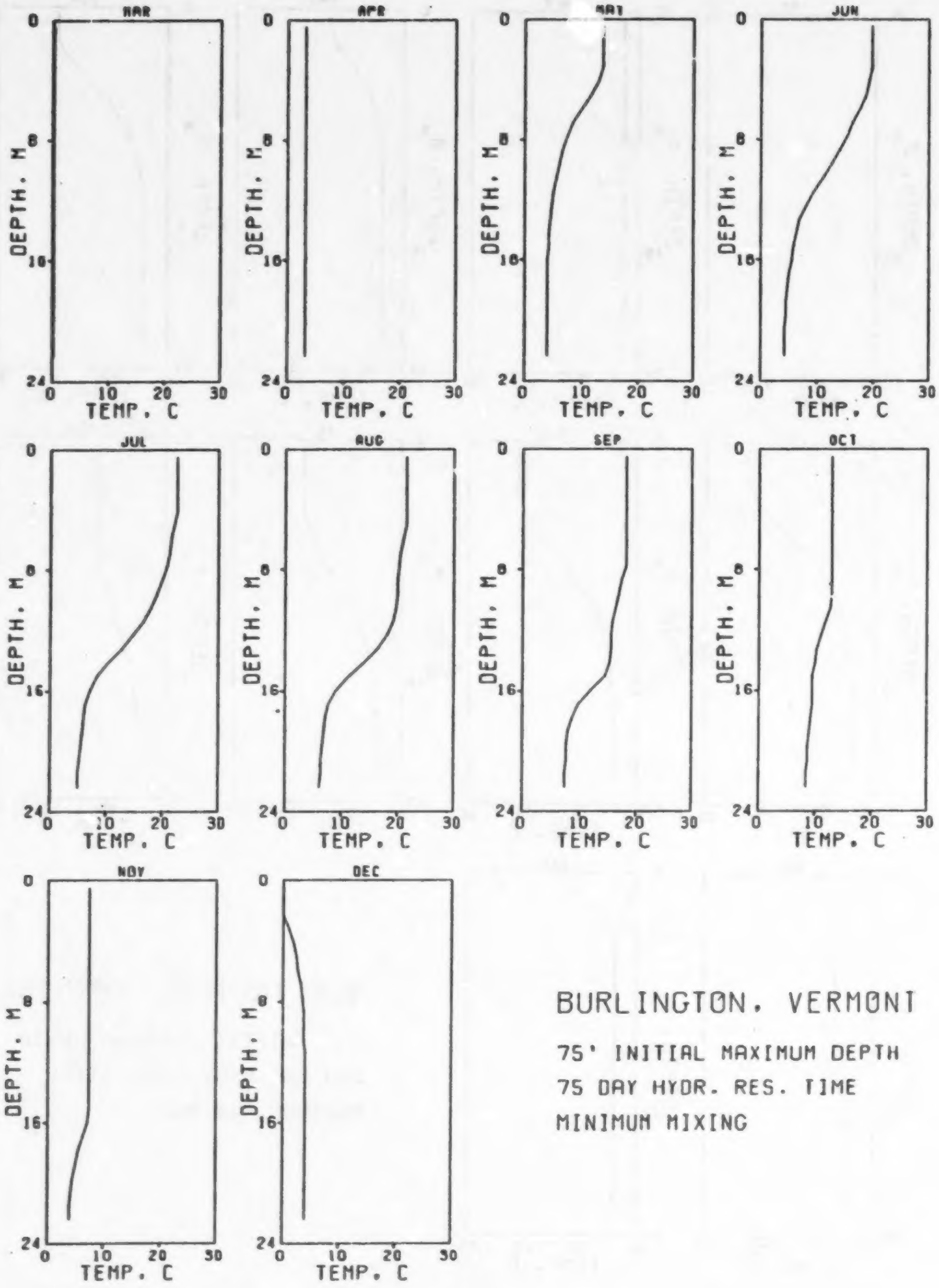






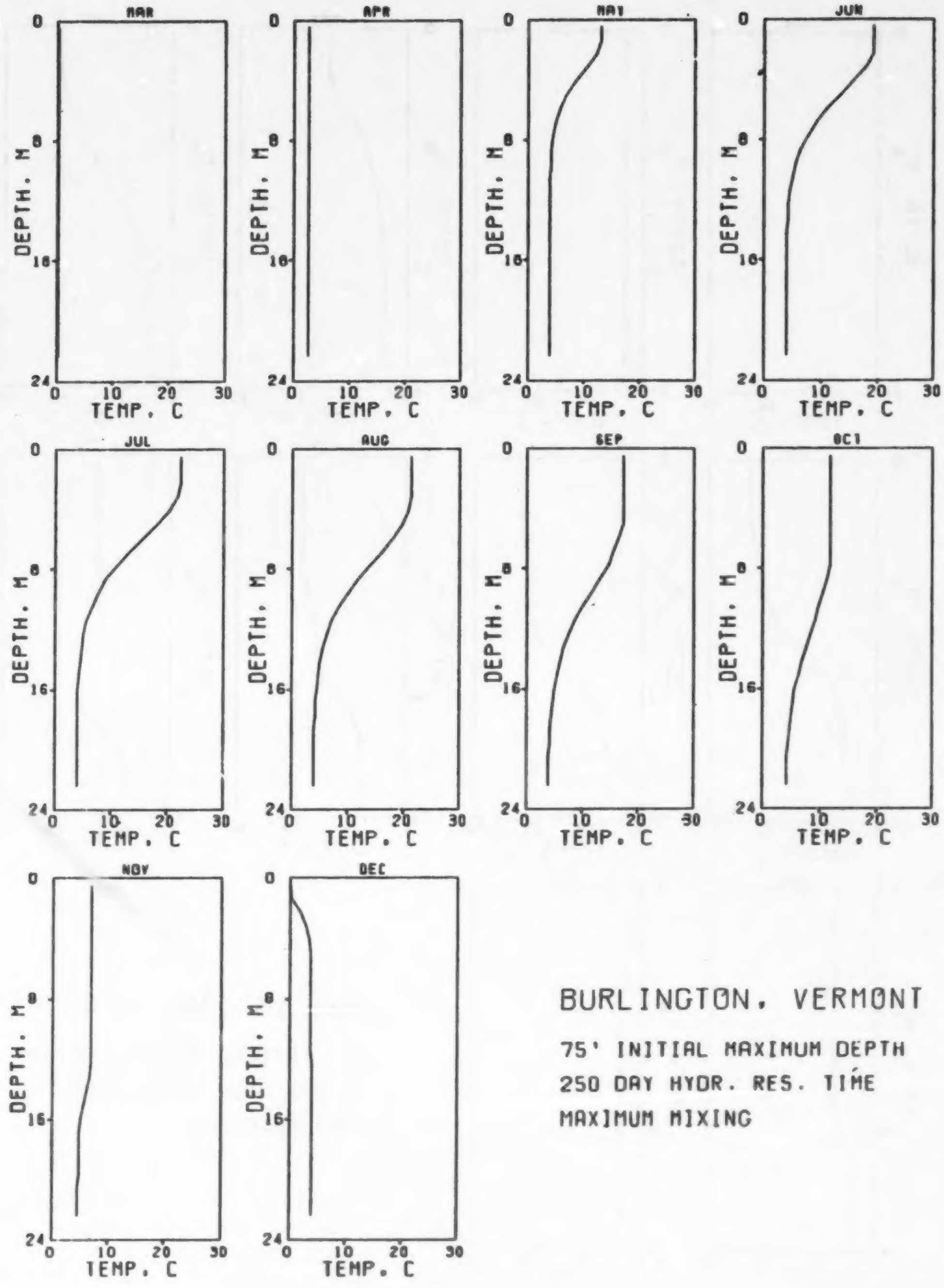
BURLINGTON, VERMONT

75' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



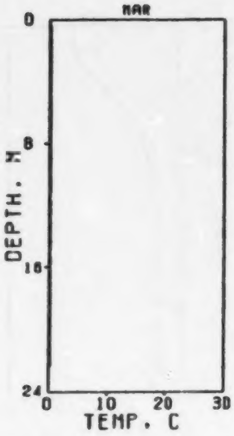
BURLINGTON, VERMONT

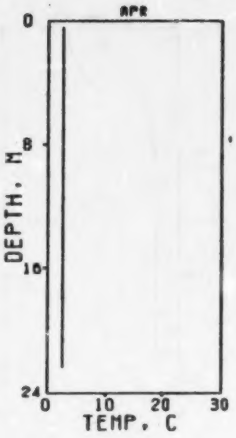
75' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING

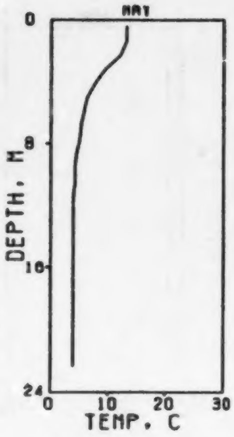


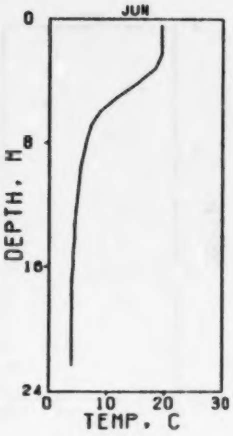
BURLINGTON, VERMONT

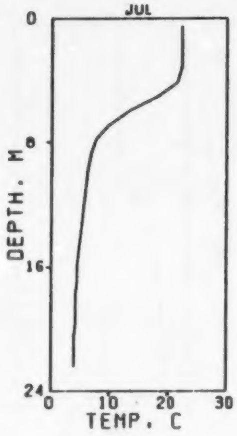
75' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING

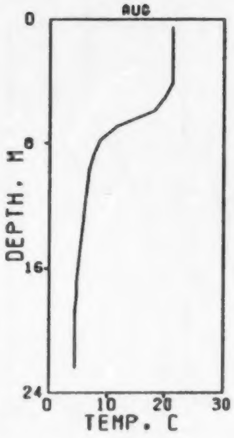


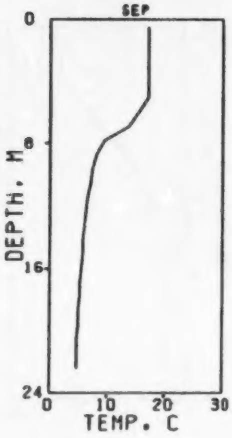


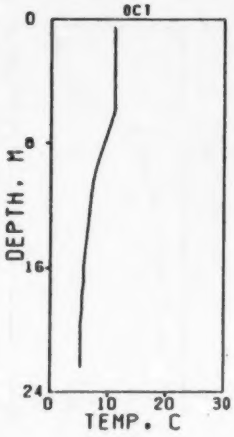


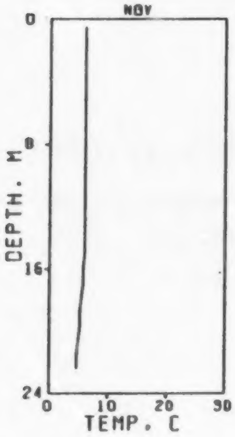


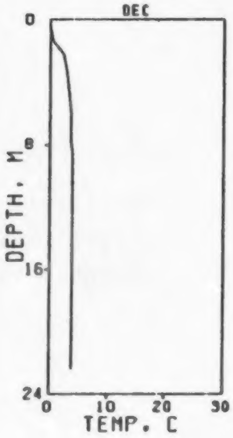










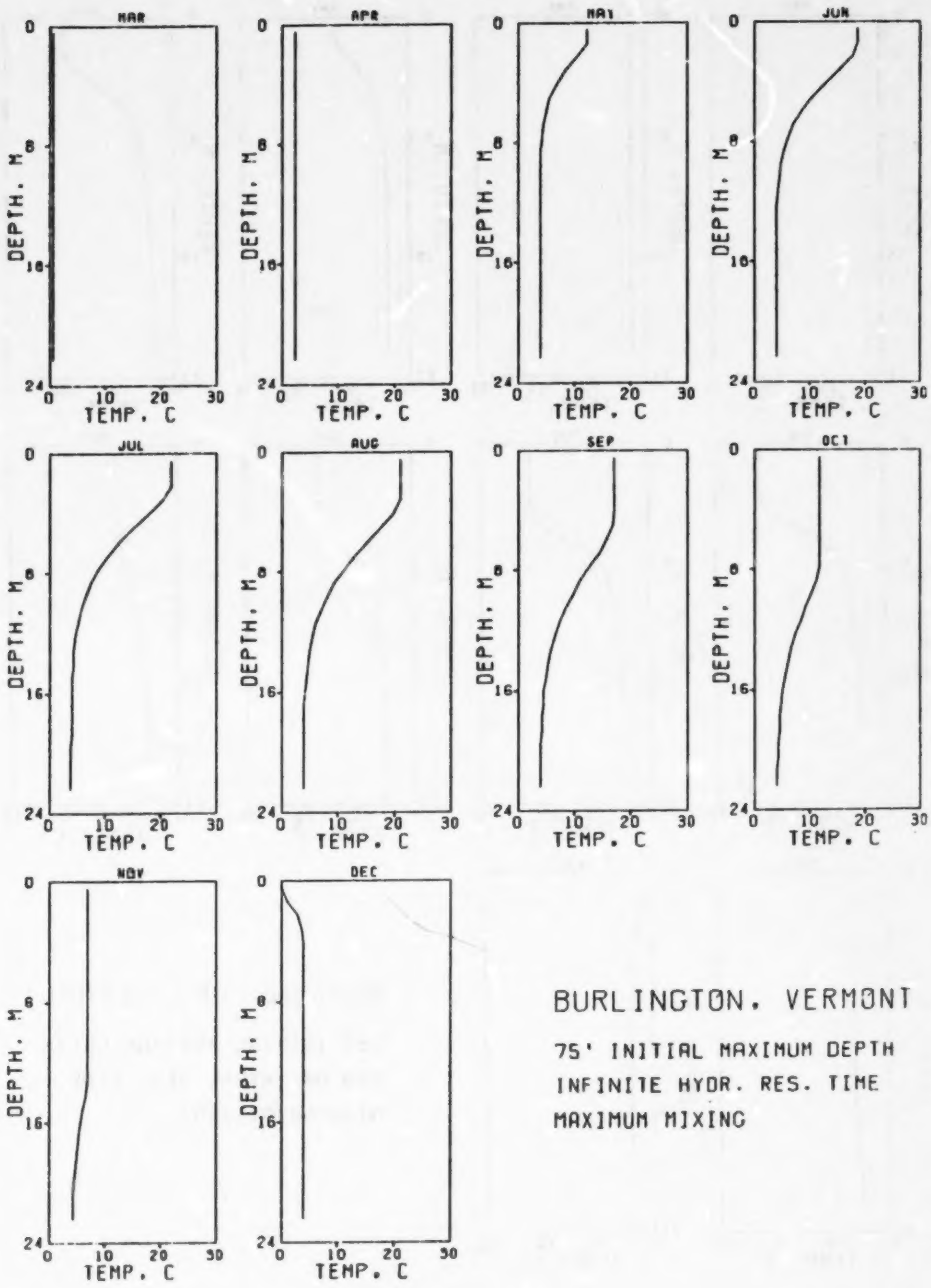


BURLINGTON, VERMONT

75' INITIAL MAXIMUM DEPTH

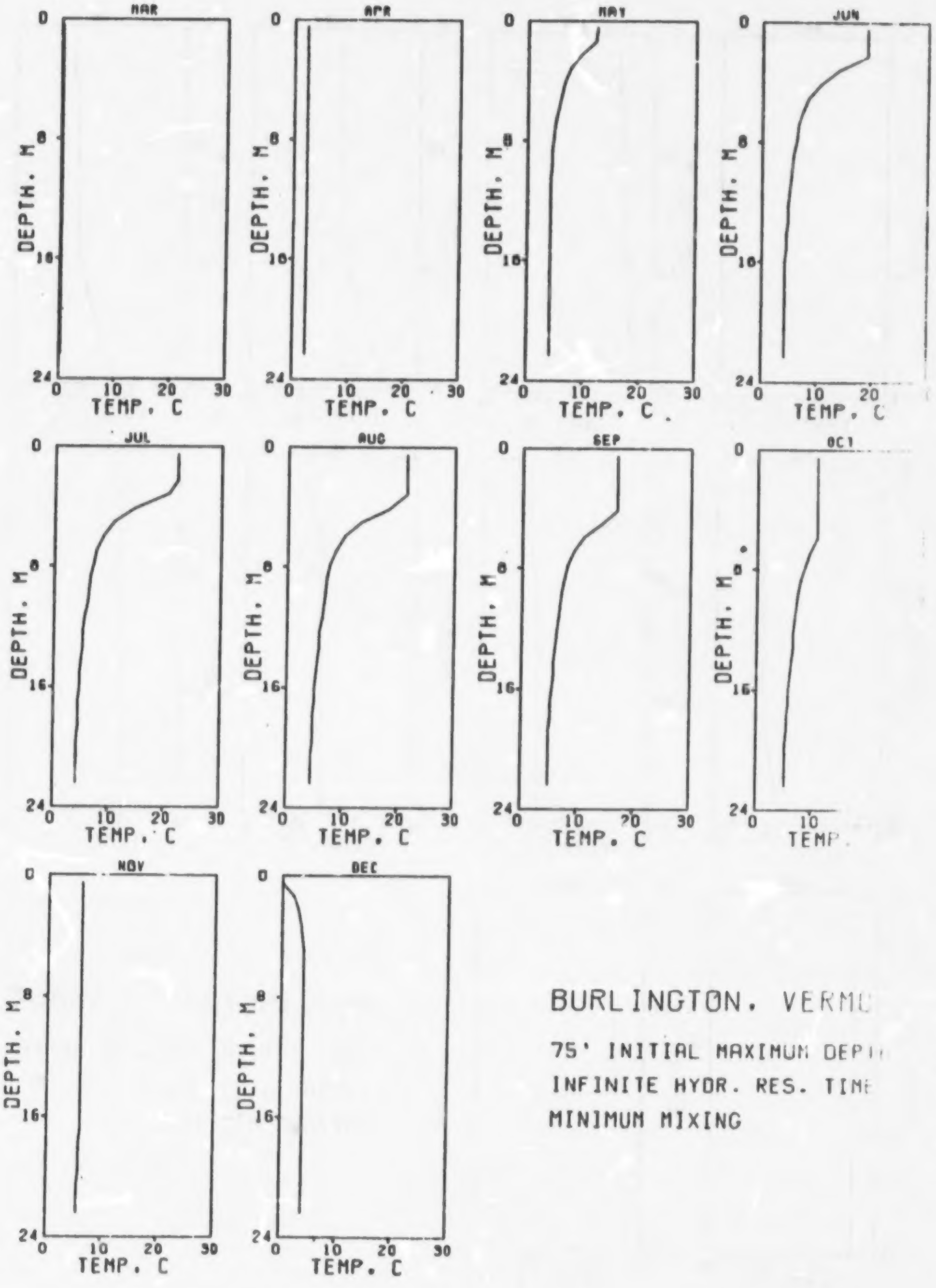
250 DAY HYDR. RES. TIME

MINIMUM MIXING



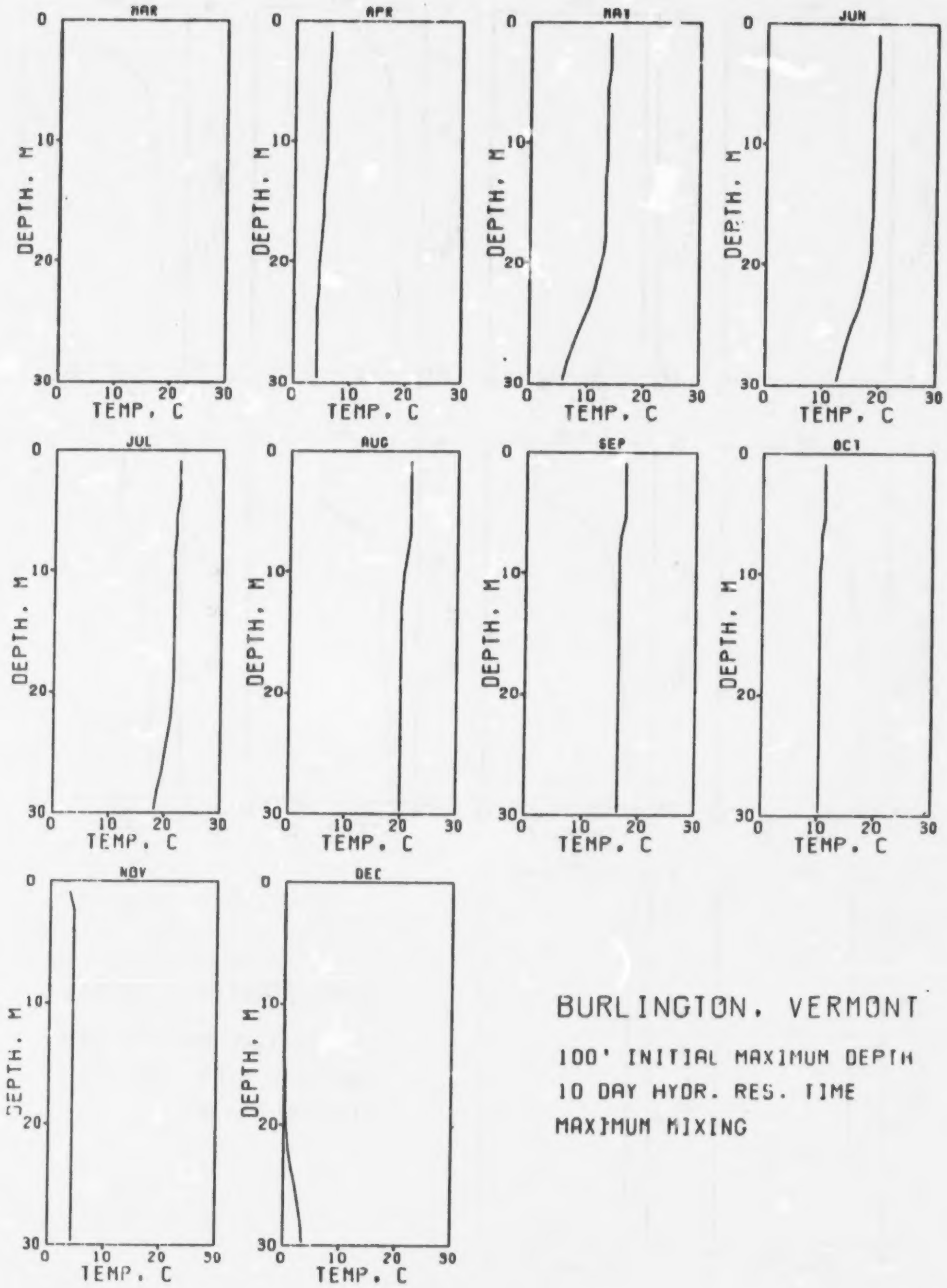
BURLINGTON, VERMONT

75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



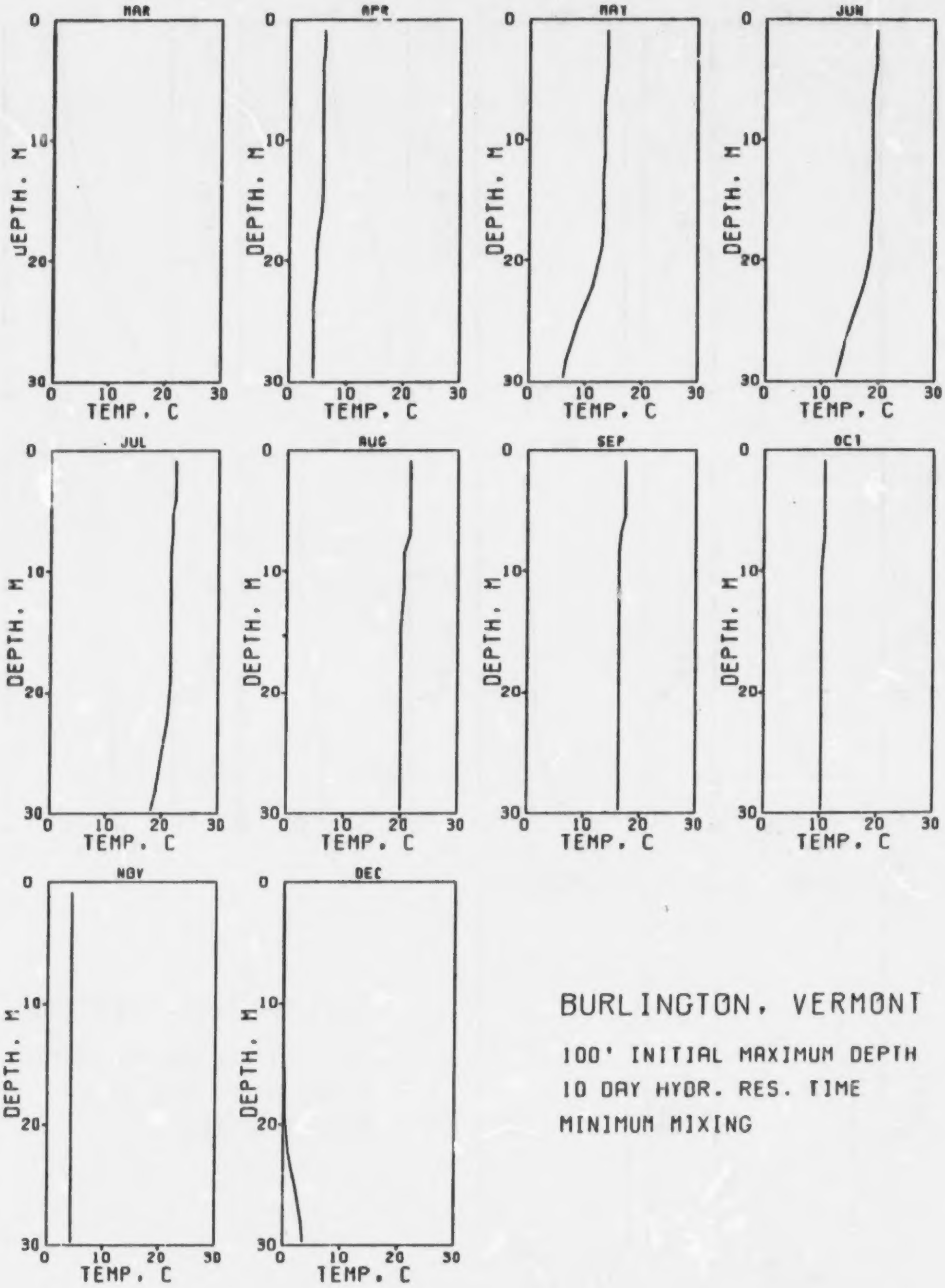
BURLINGTON, VERM

75' INITIAL MAXIMUM DEP
INFINITE HYDR. RES. TIM
MINIMUM MIXING



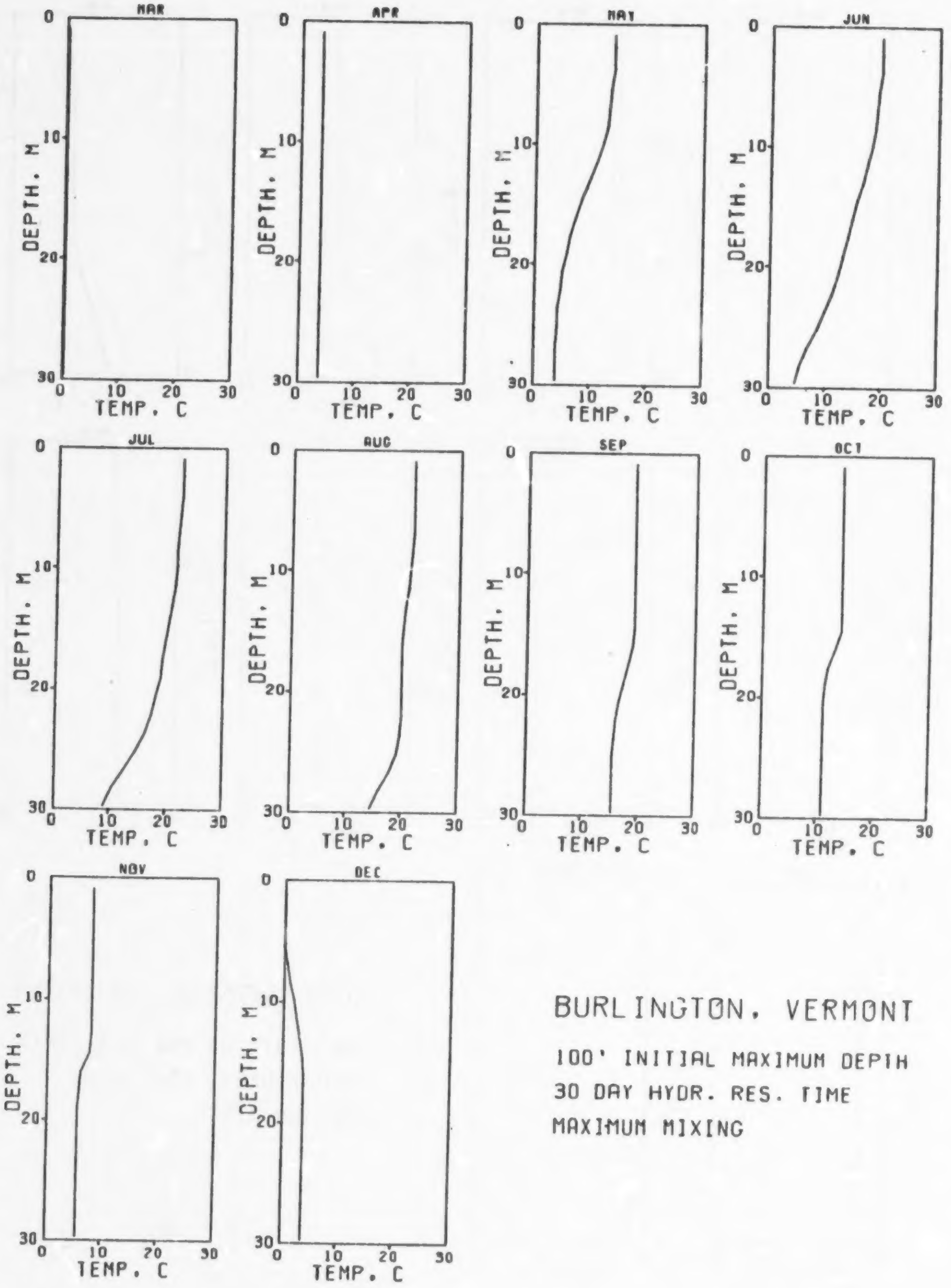
BURLINGTON, VERMONT

100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



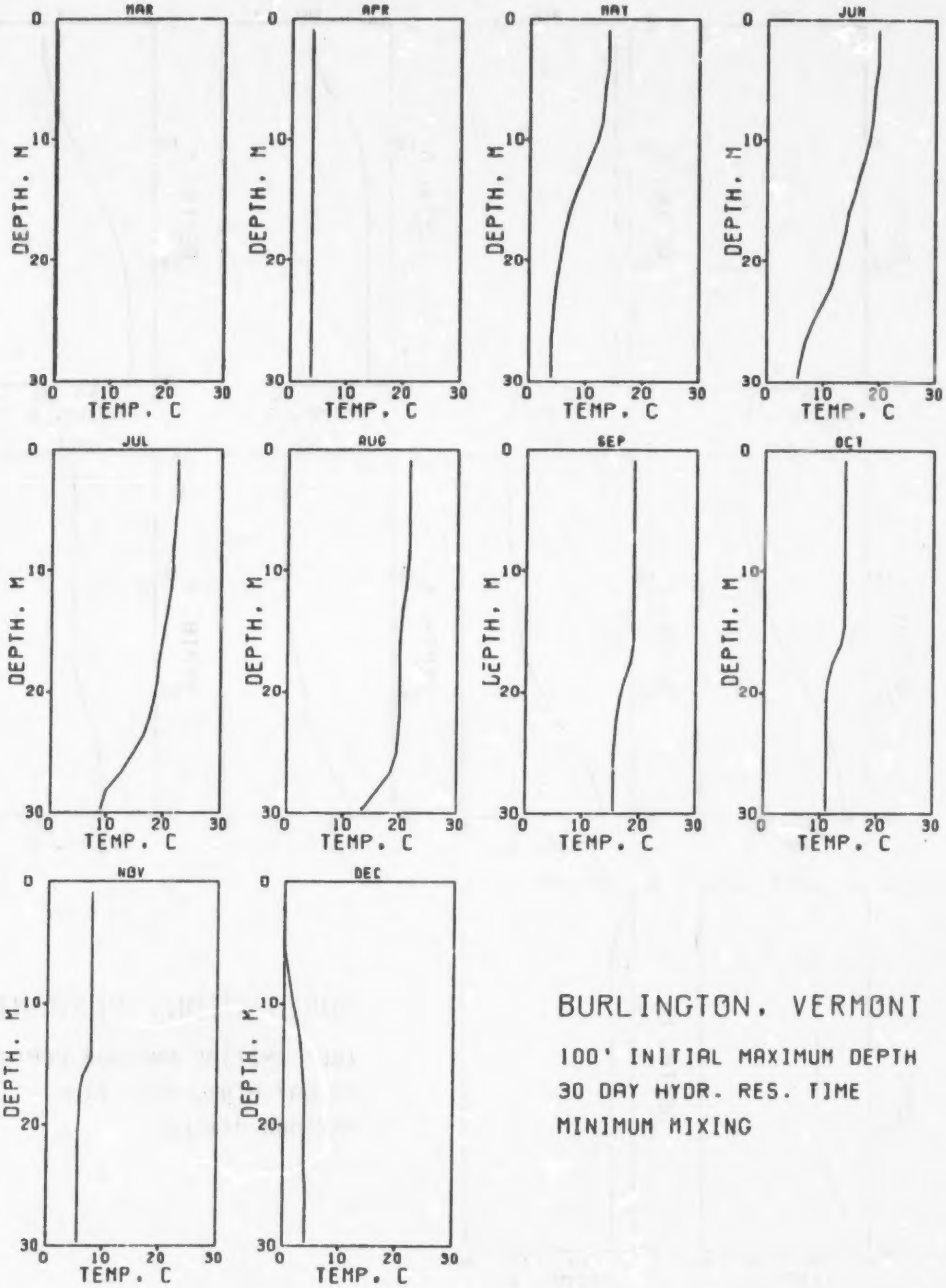
BURLINGTON, VERMONT

100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



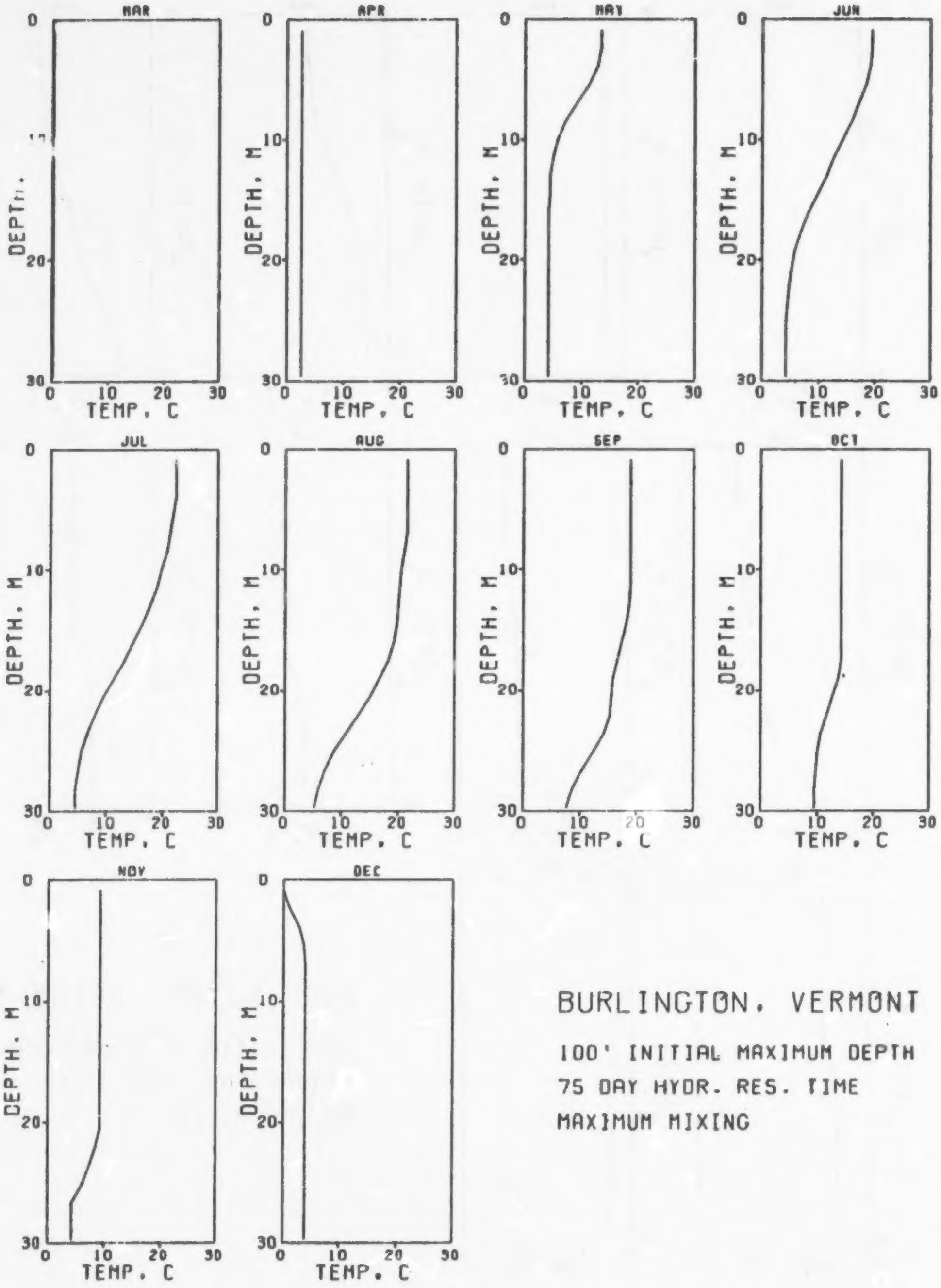
BURLINGTON, VERMONT

100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



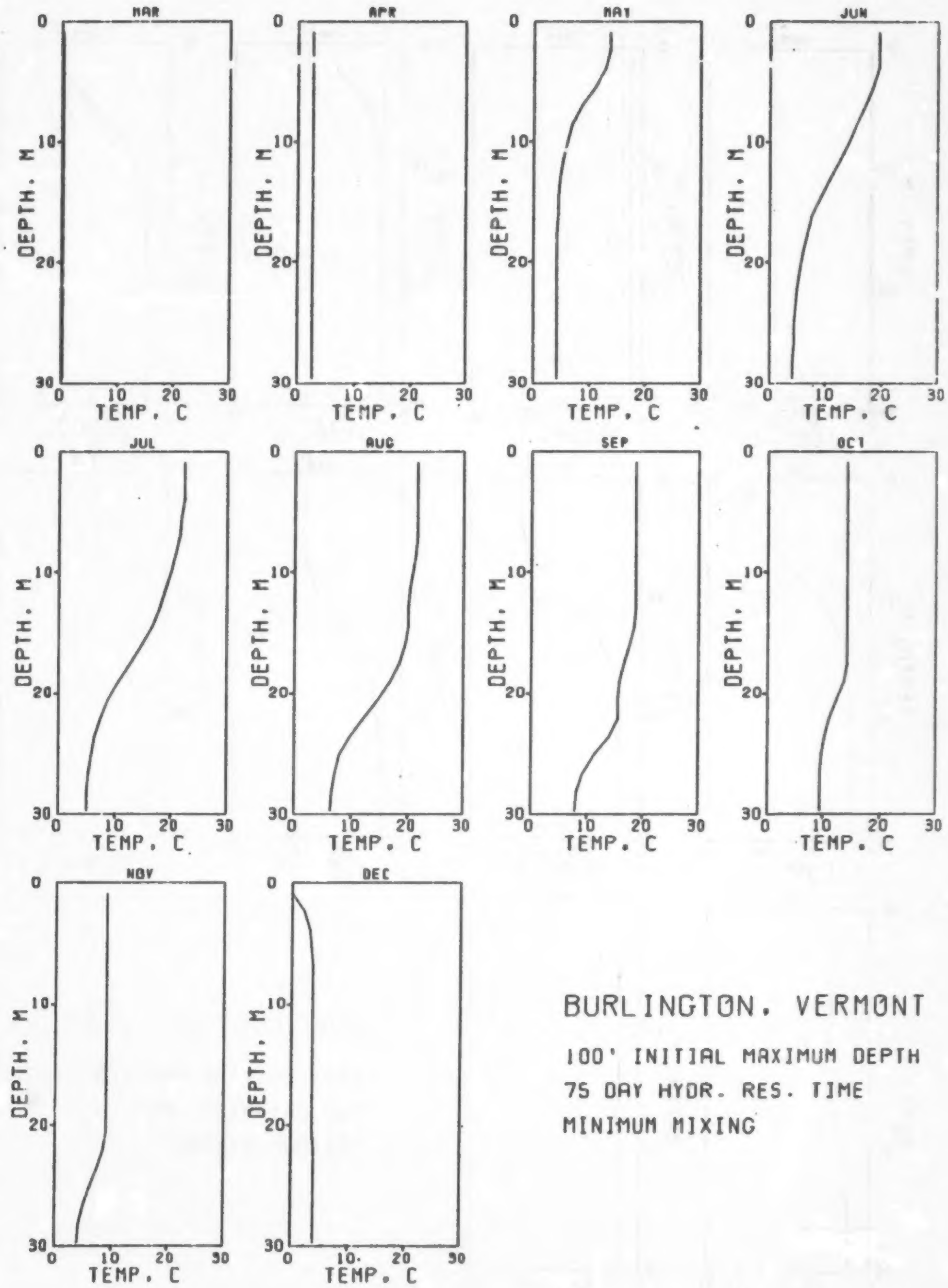
BURLINGTON, VERMONT

100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



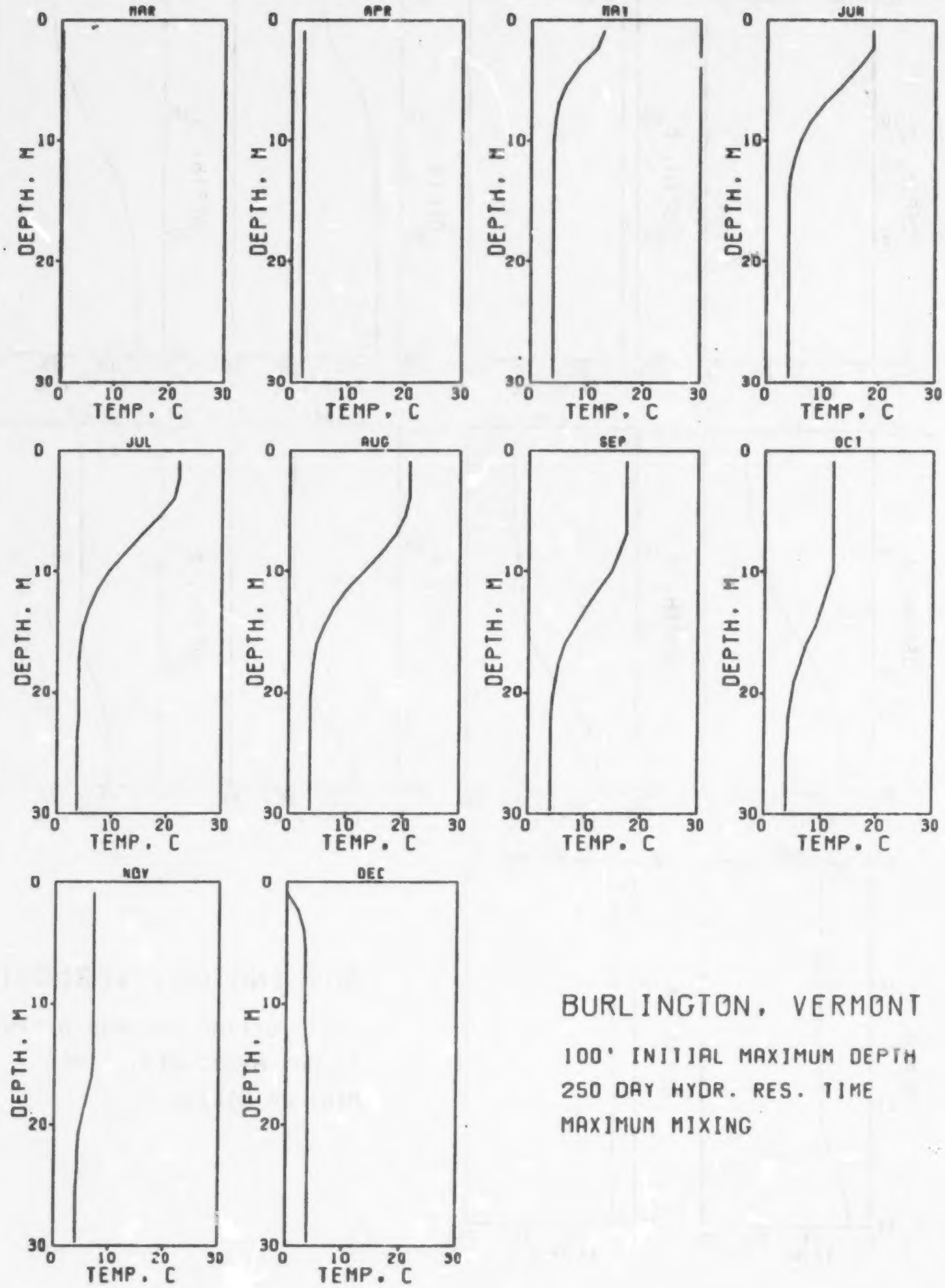
BURLINGTON, VERMONT

100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



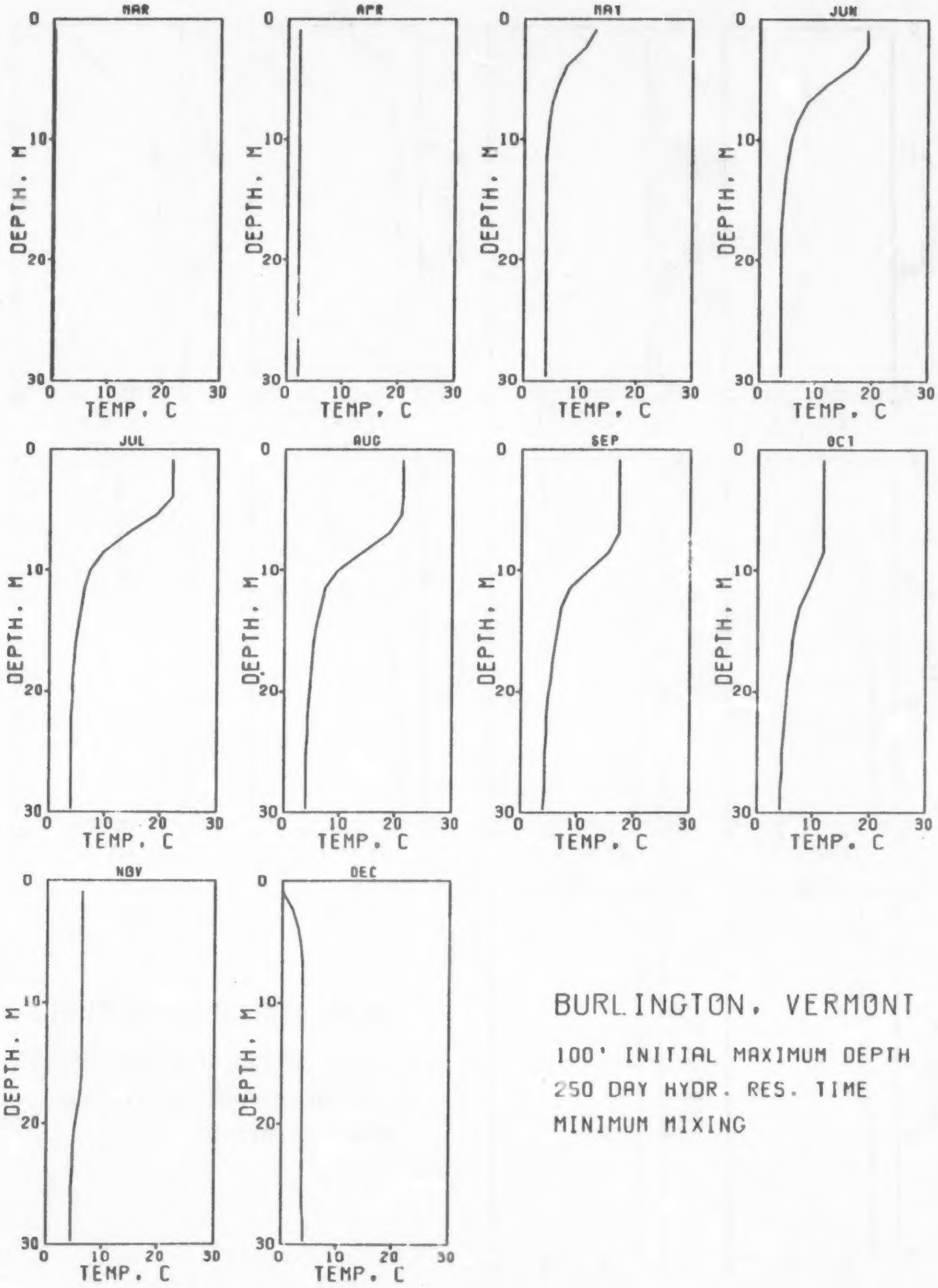
BURLINGTON, VERMONT

100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



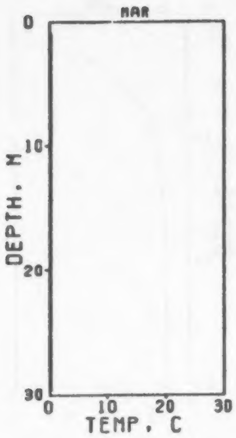
BURLINGTON, VERMONT

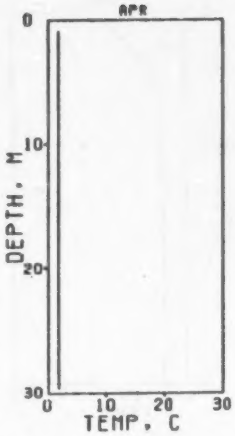
100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING

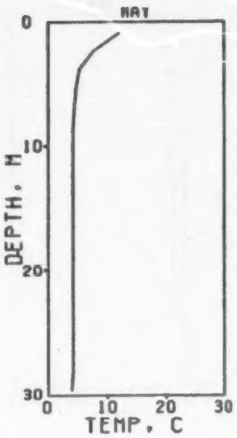


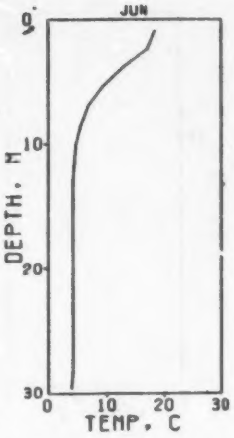
BURLINGTON, VERMONT

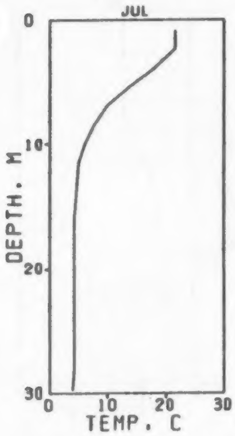
100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING

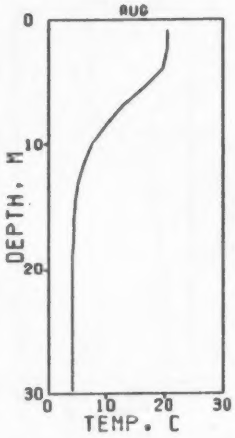


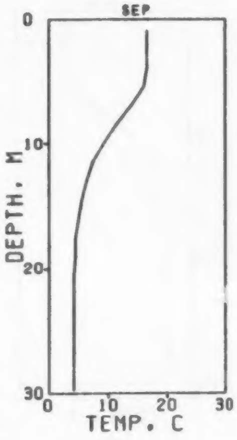


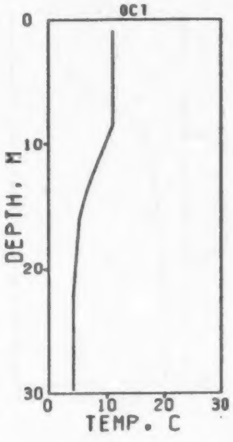


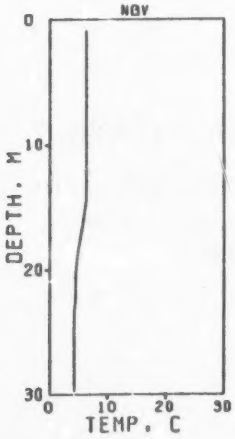


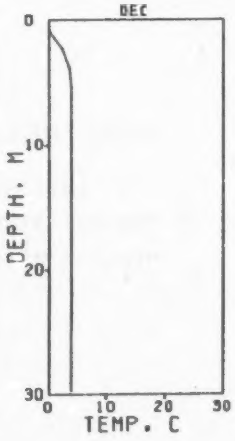






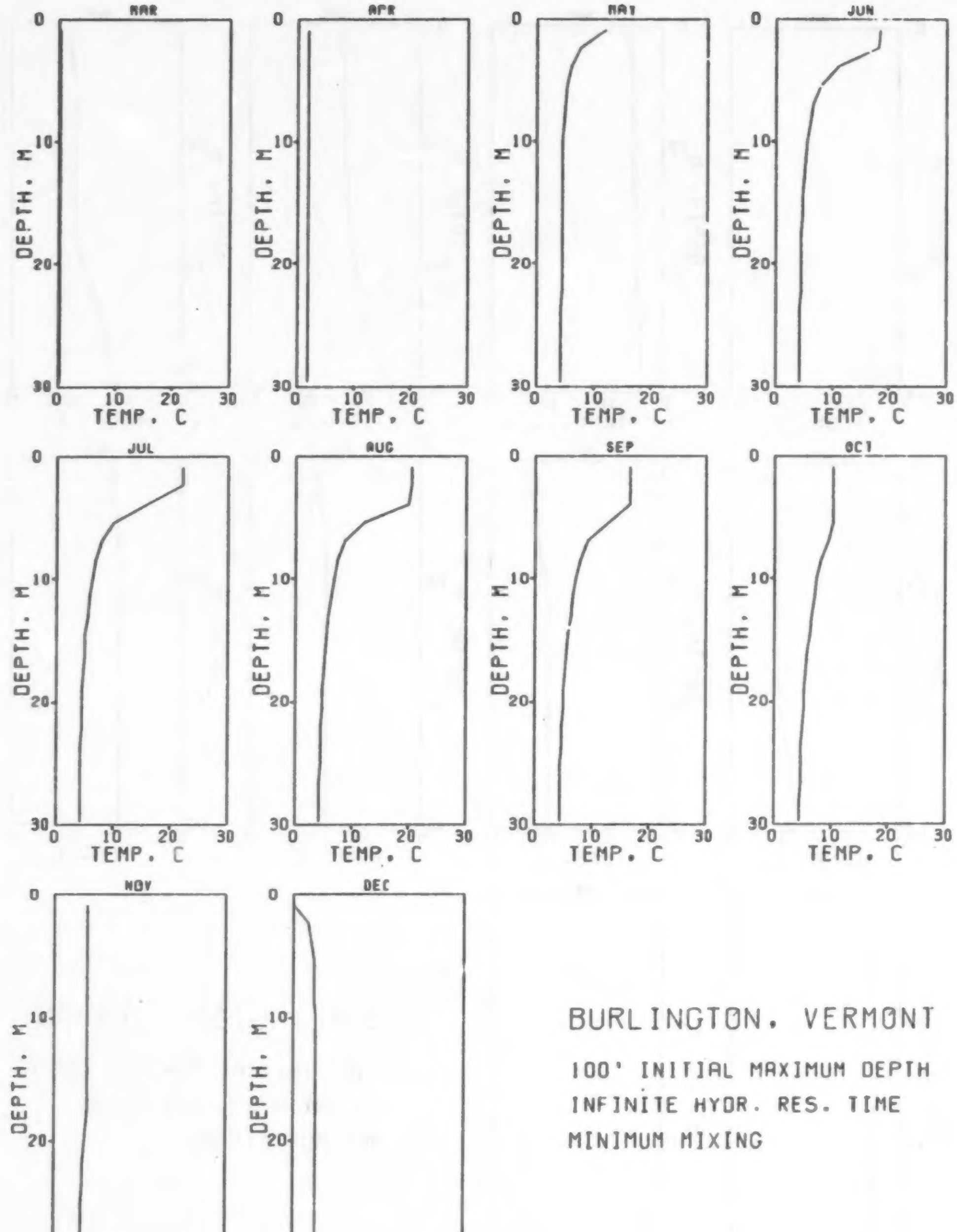






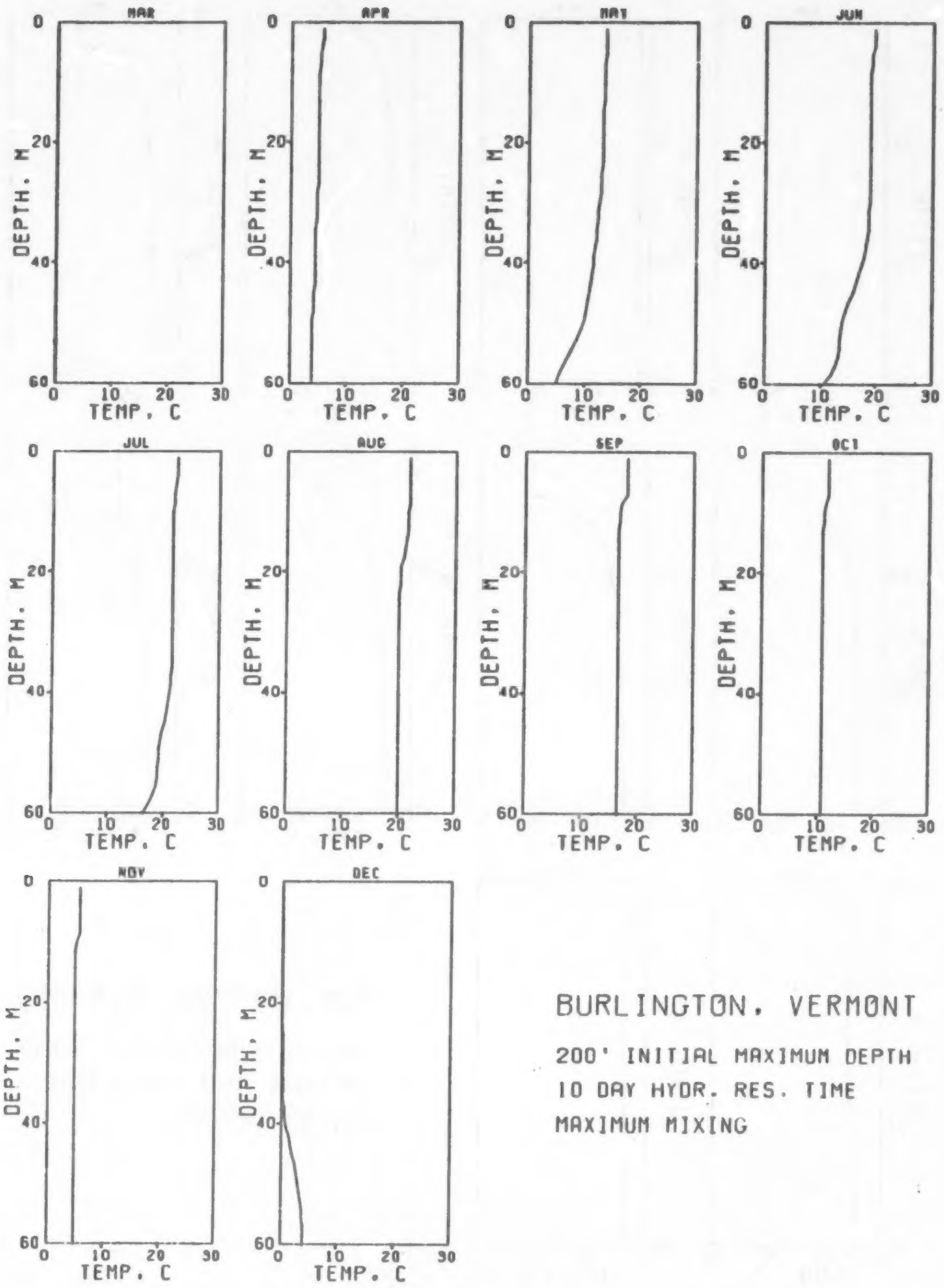
BURLINGTON, VERMONT

100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



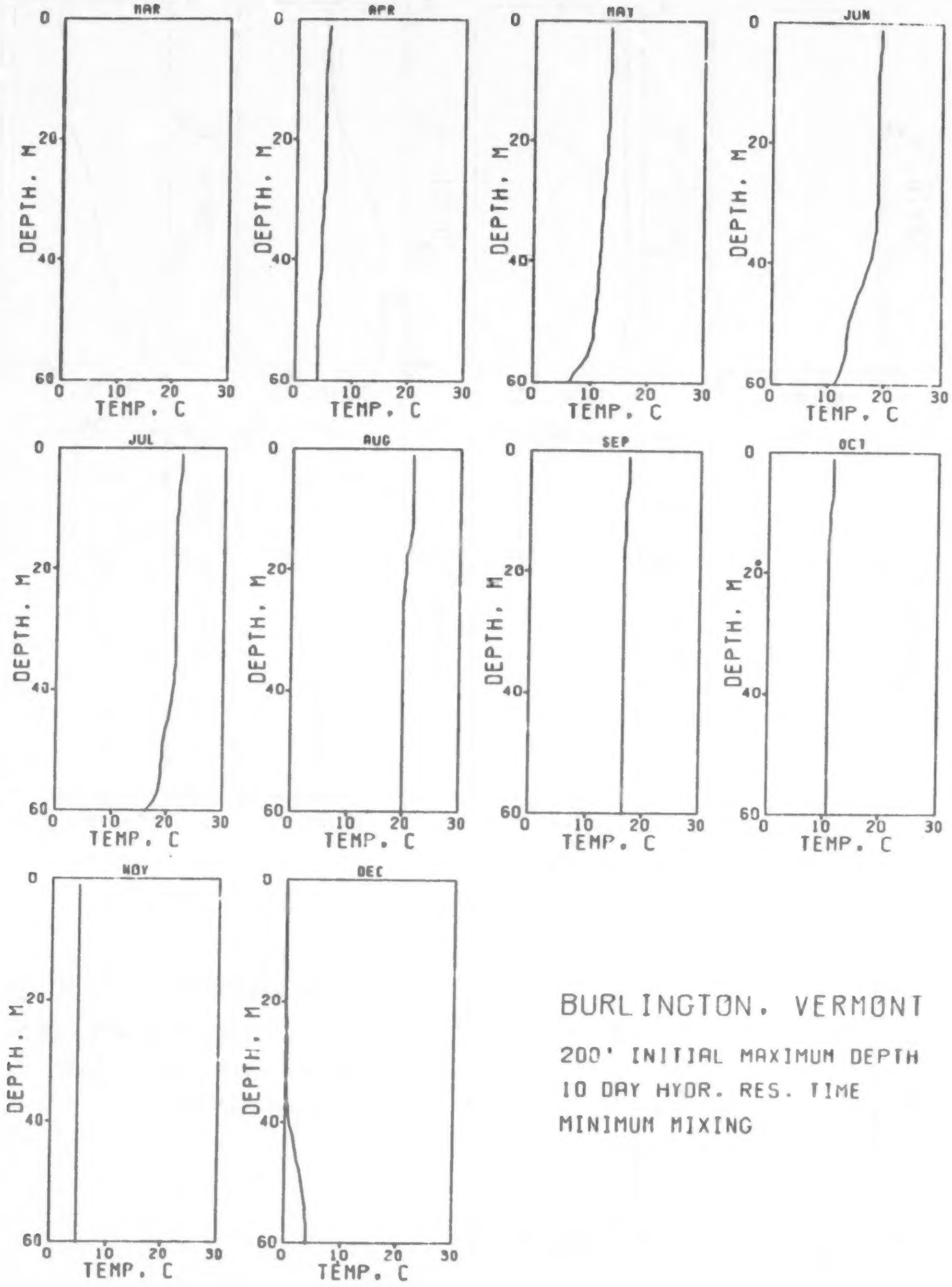
BURLINGTON, VERMONT

100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



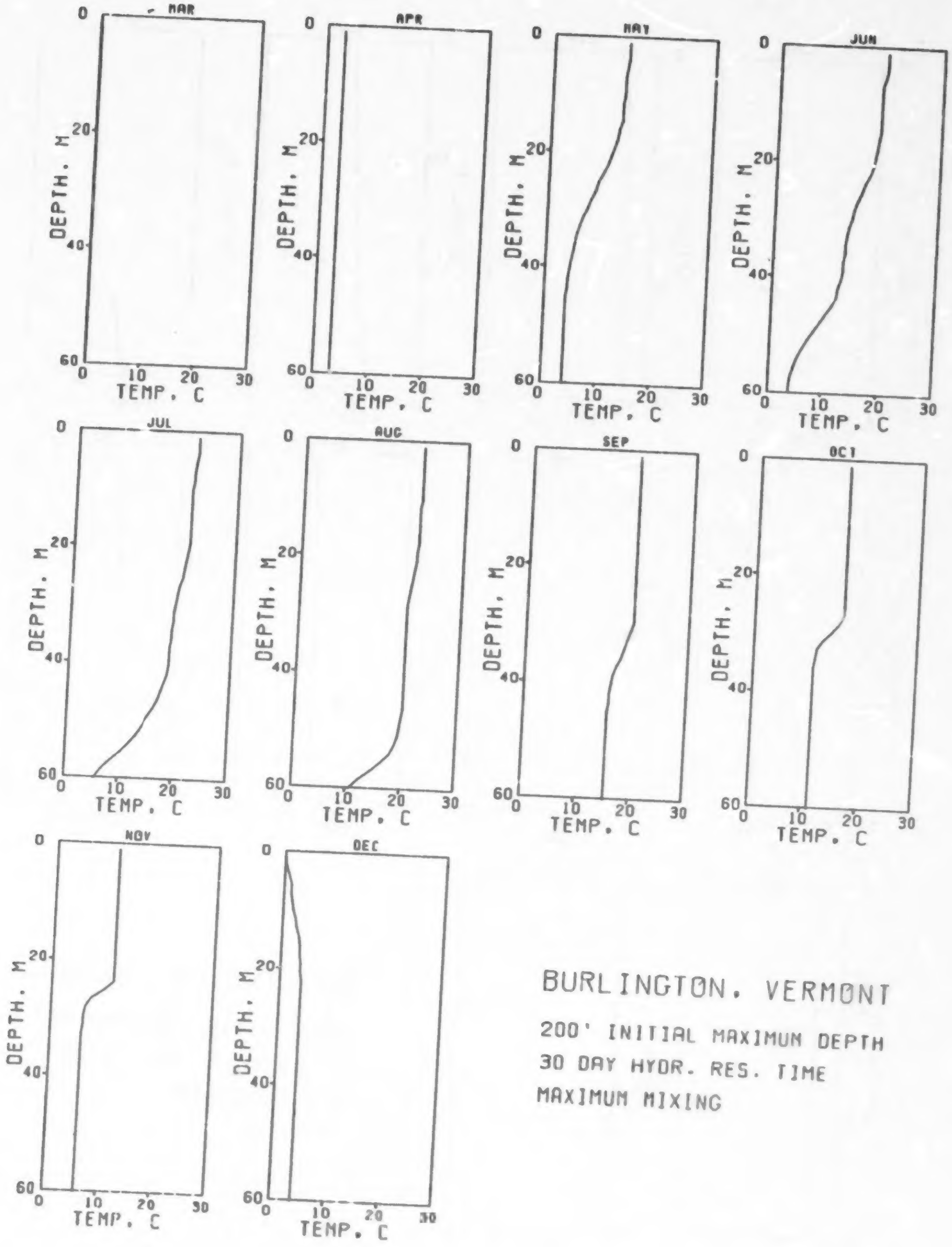
BURLINGTON, VERMONT

200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



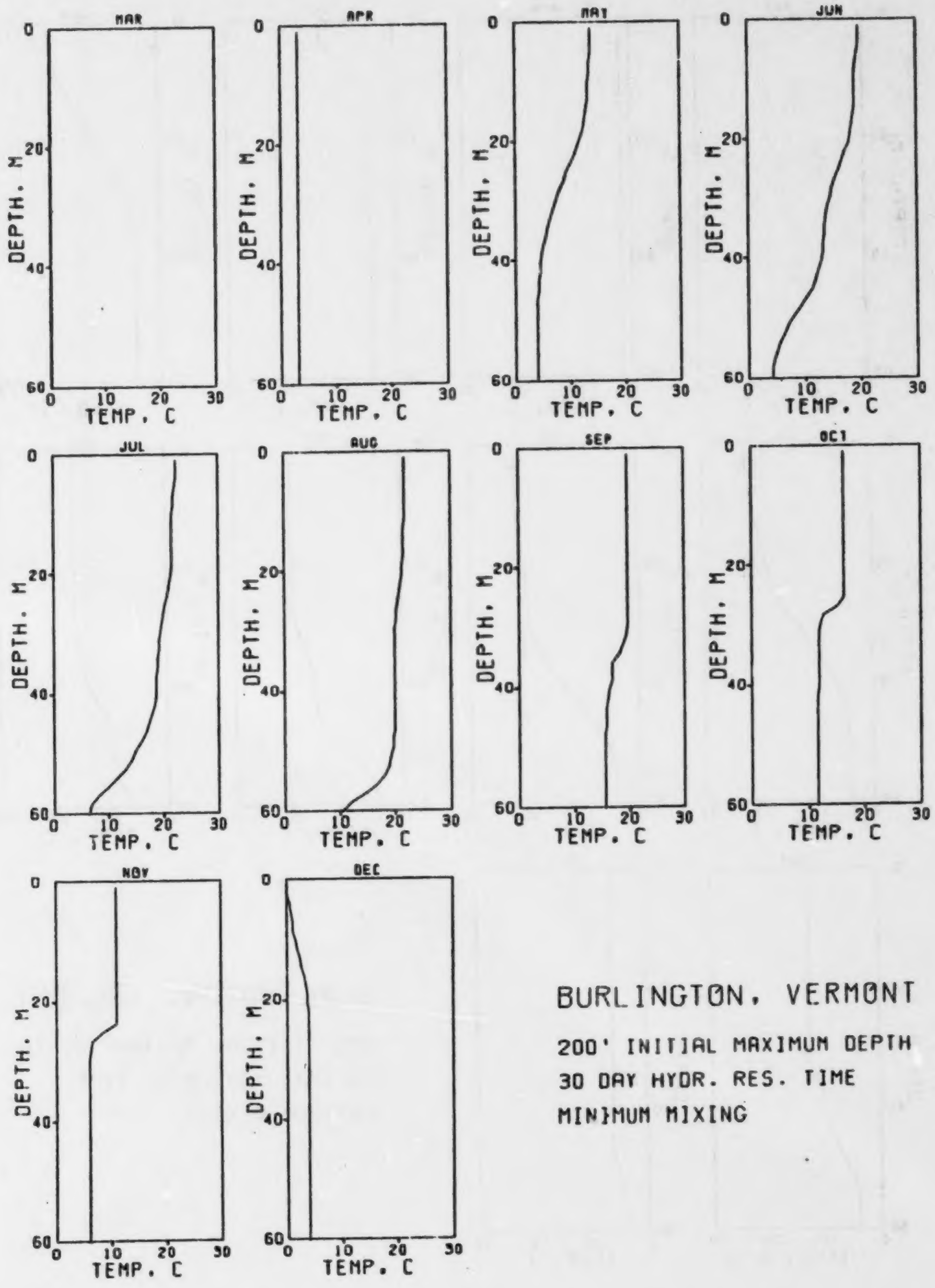
BURLINGTON, VERMONT

200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



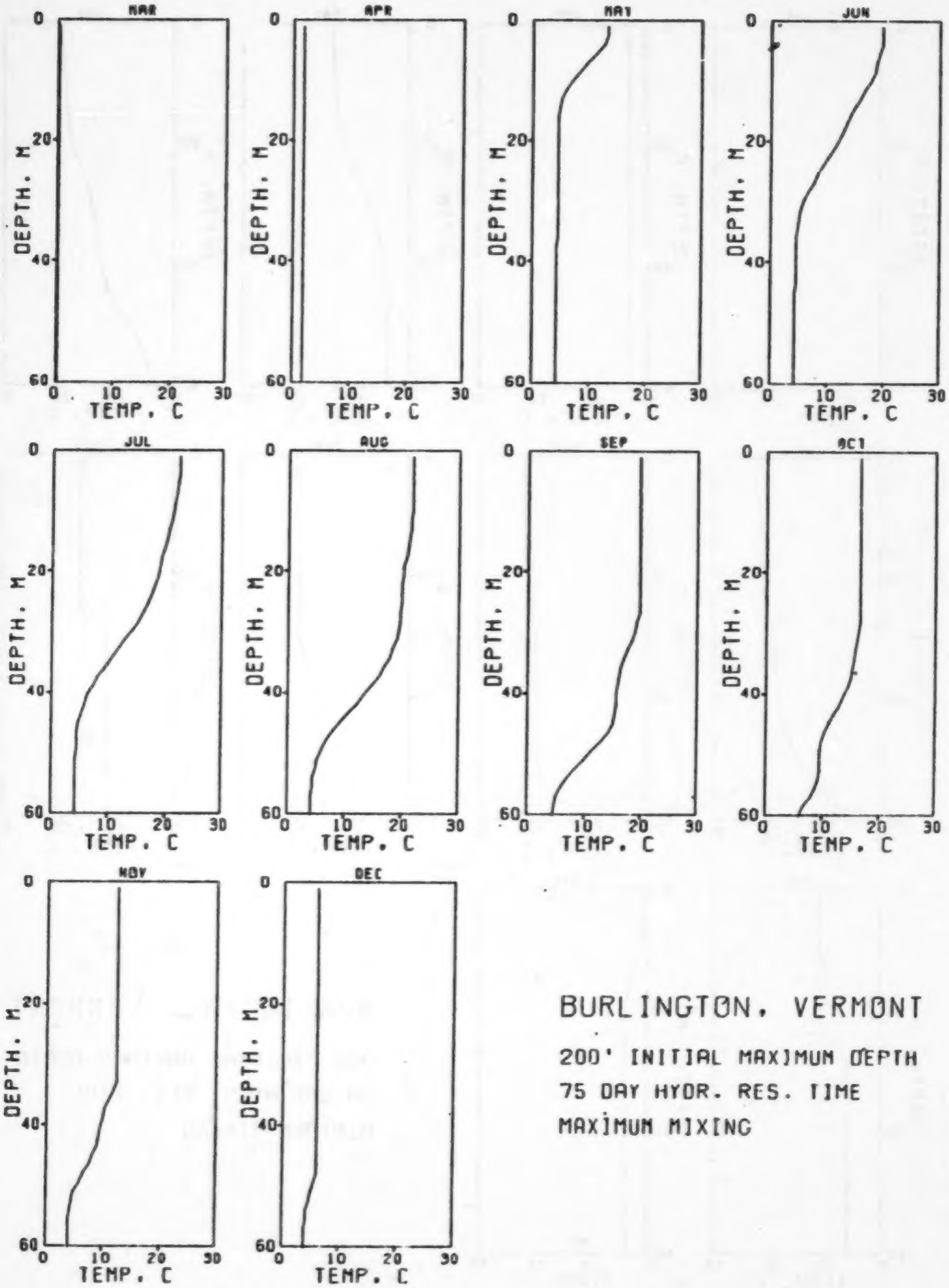
BURLINGTON, VERMONT

200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



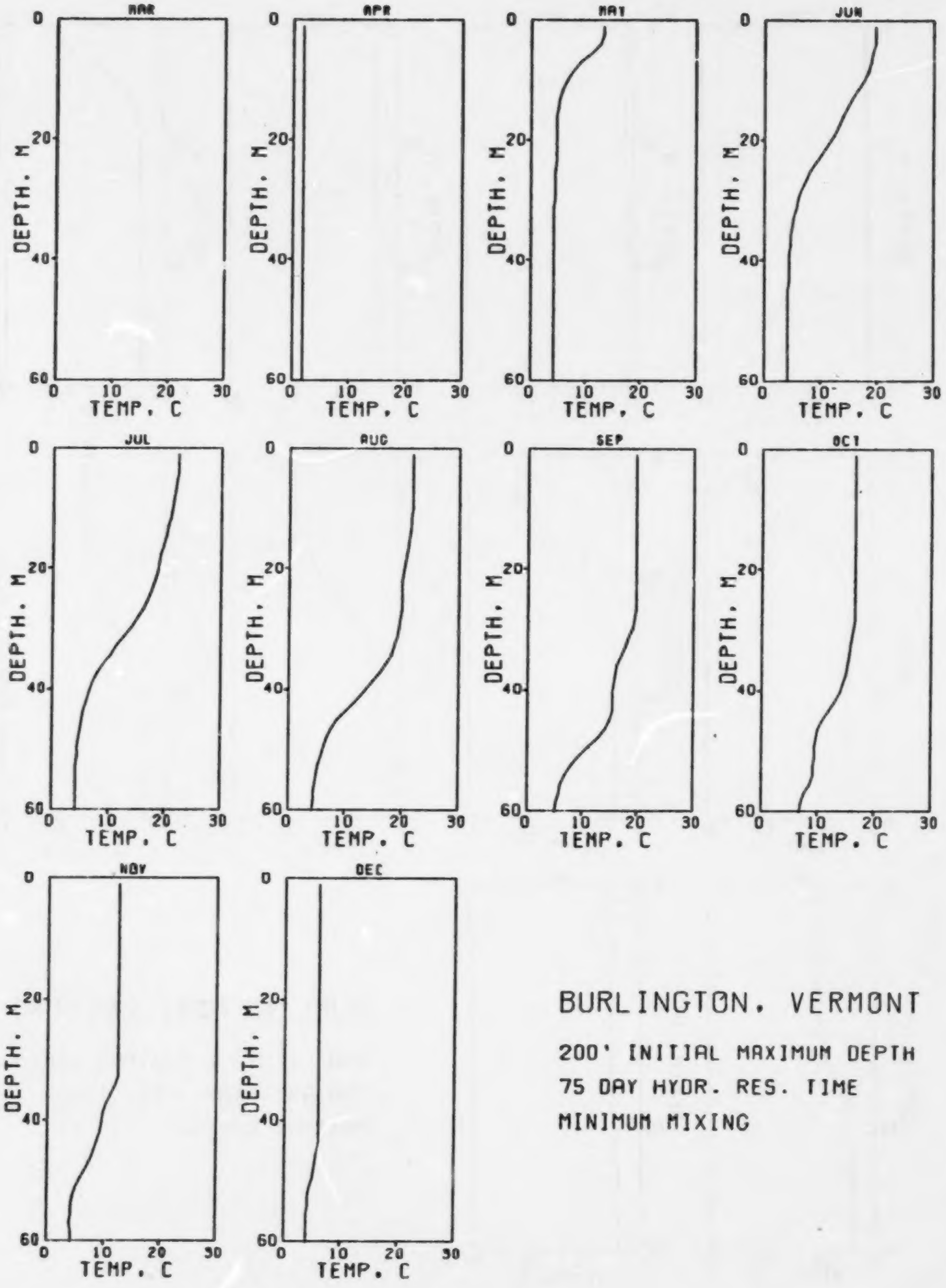
BURLINGTON, VERMONT

200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



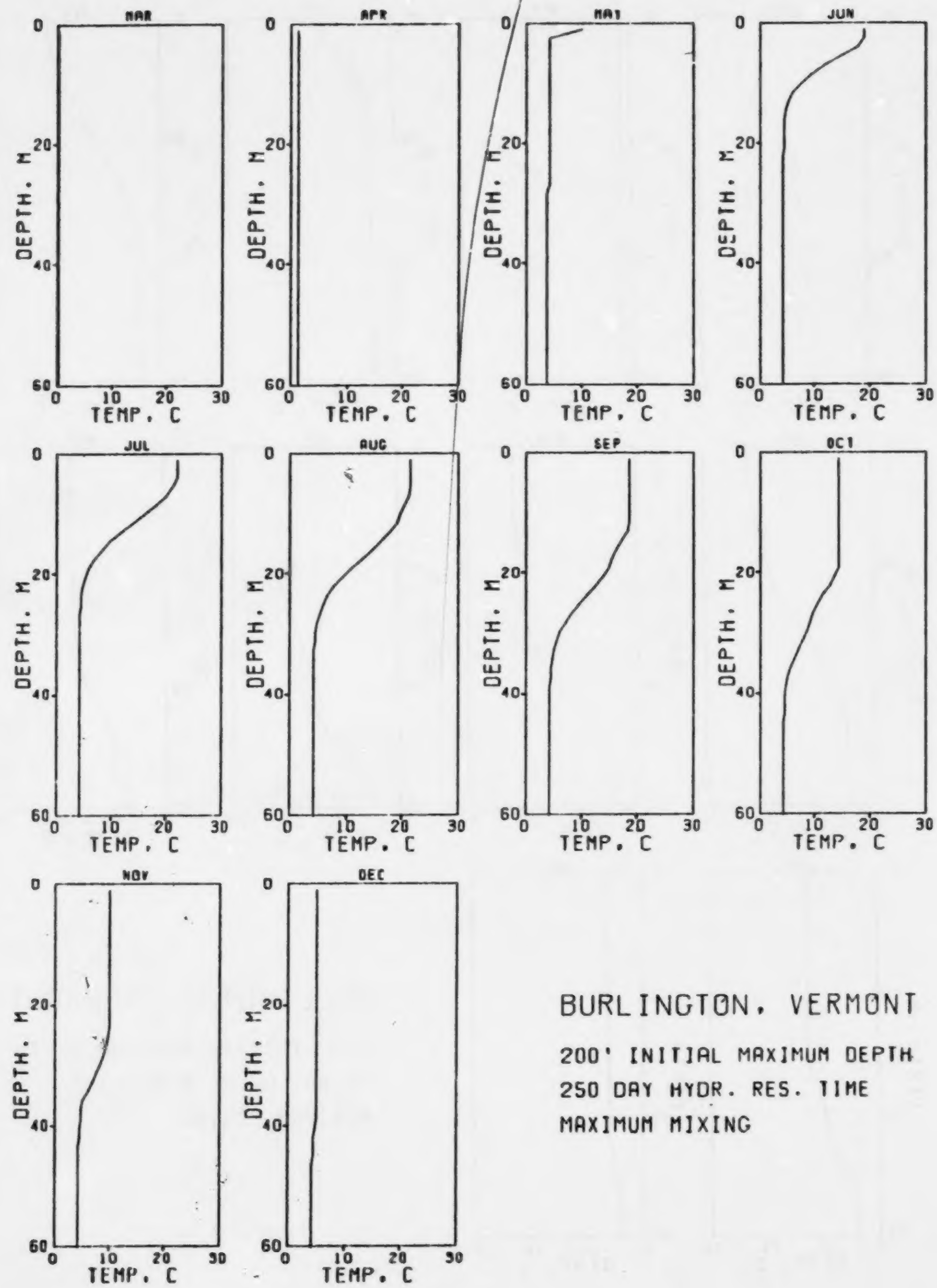
BURLINGTON, VERMONT

200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



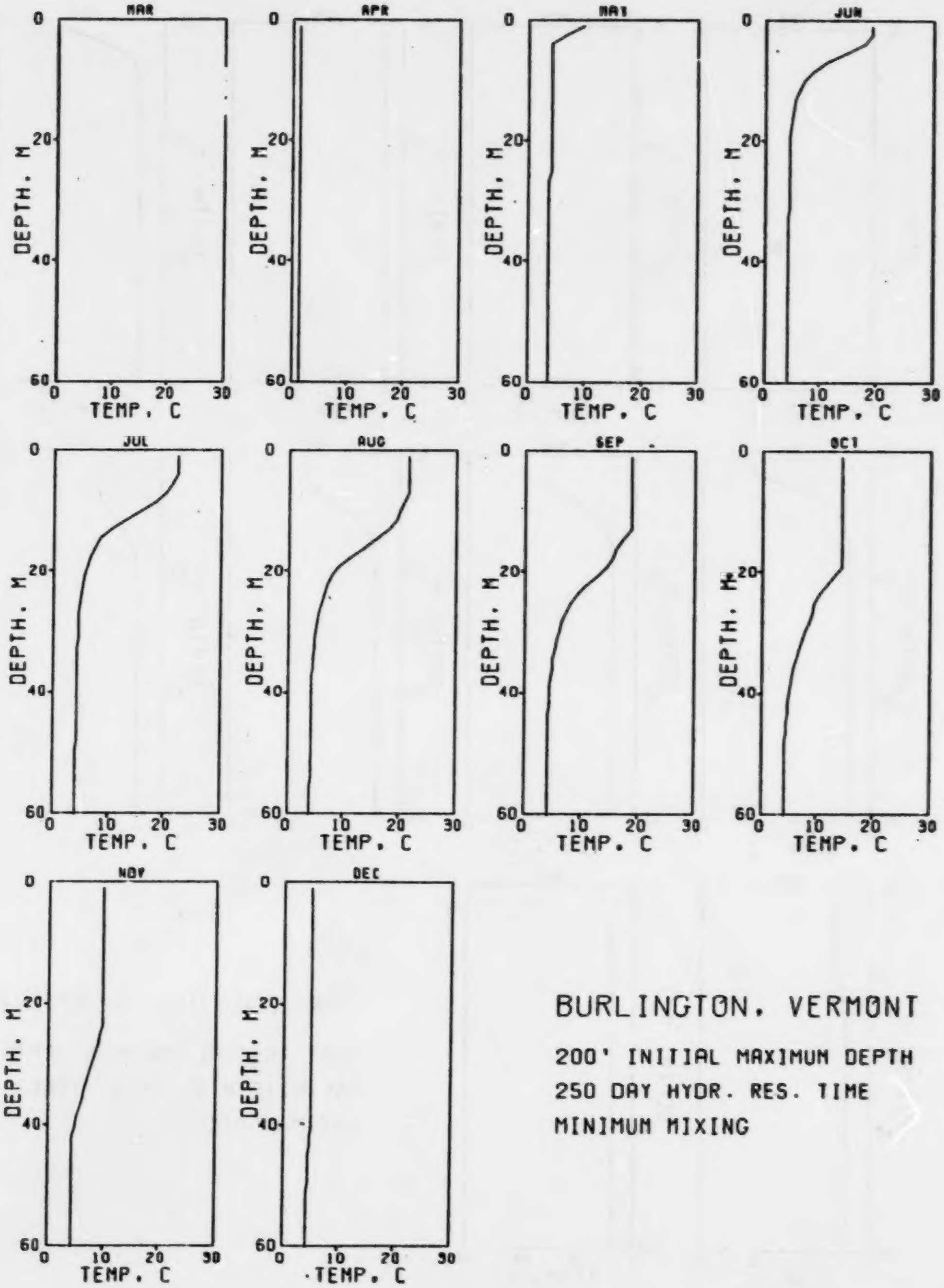
BURLINGTON, VERMONT

200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



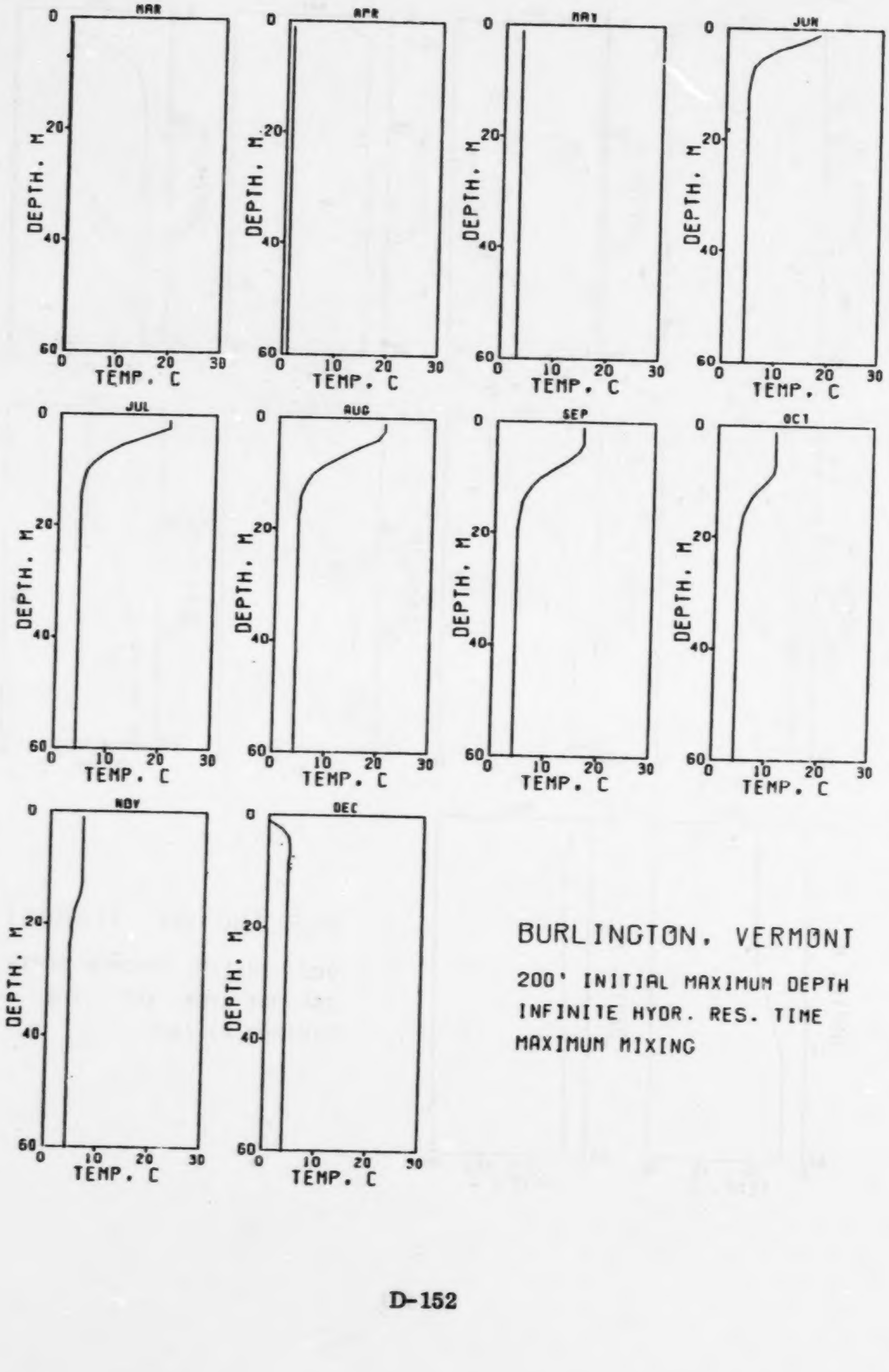
BURLINGTON, VERMONT

200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



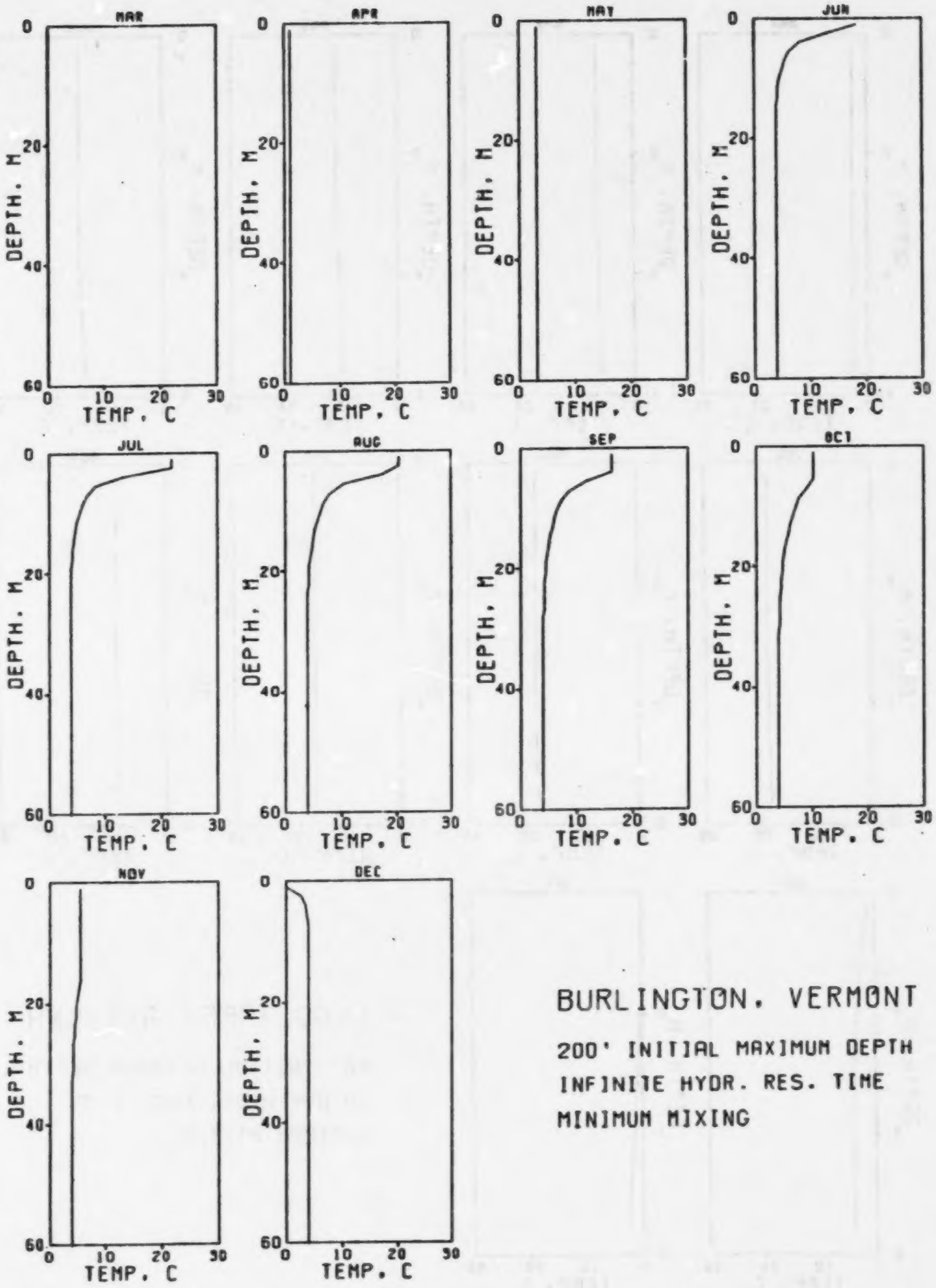
BURLINGTON, VERMONT

200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



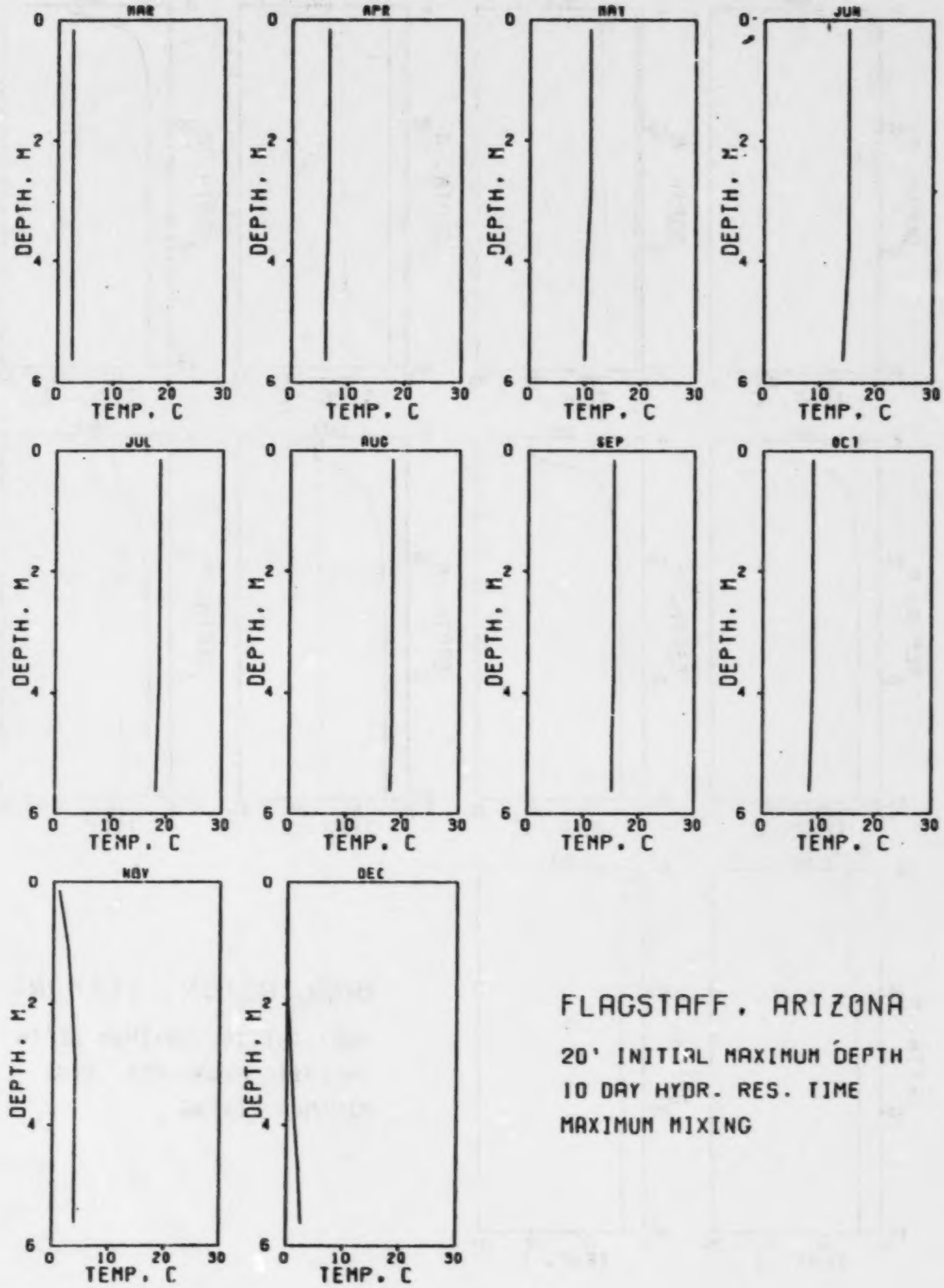
BURLINGTON, VERMONT

200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



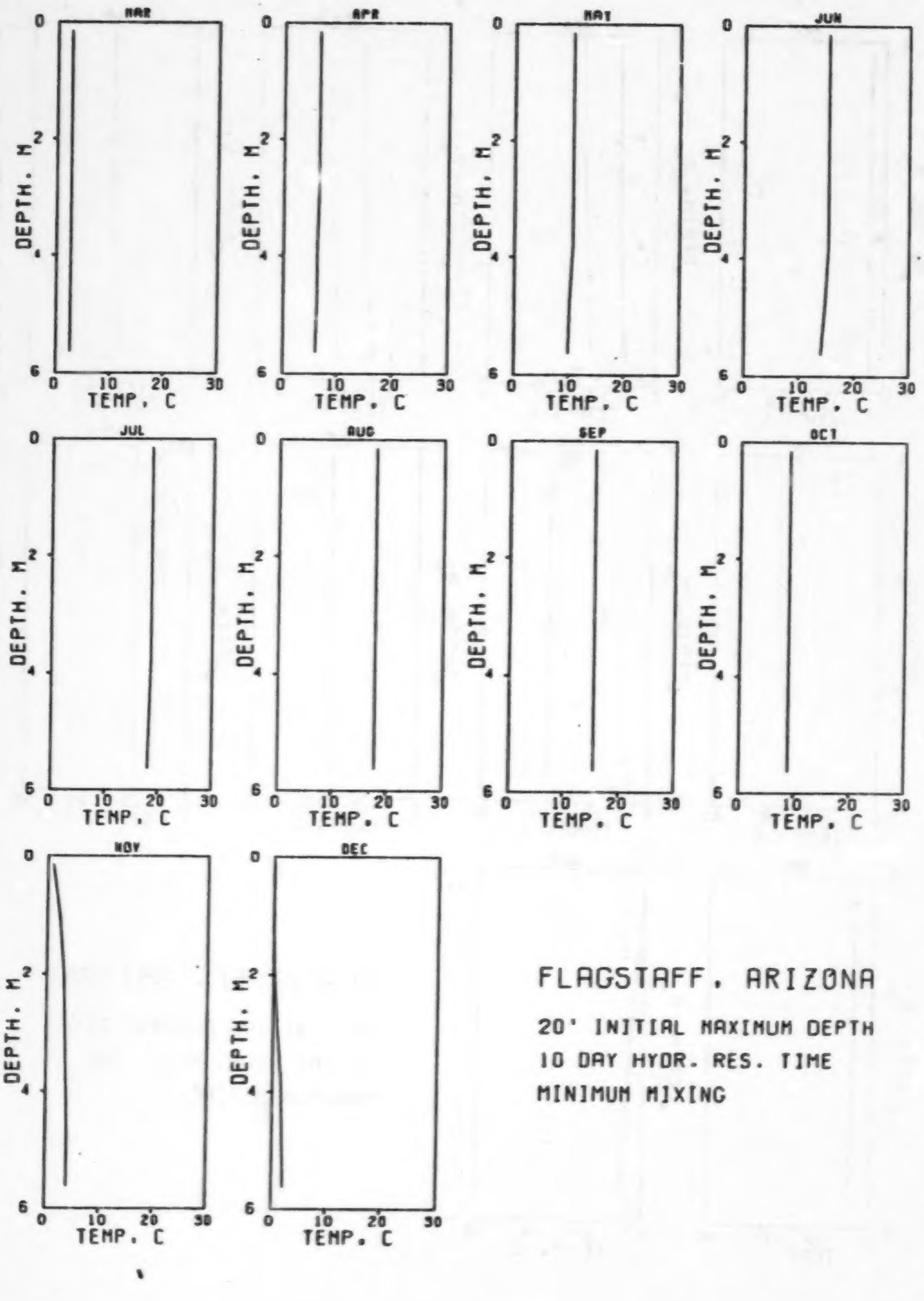
BURLINGTON, VERMONT

200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



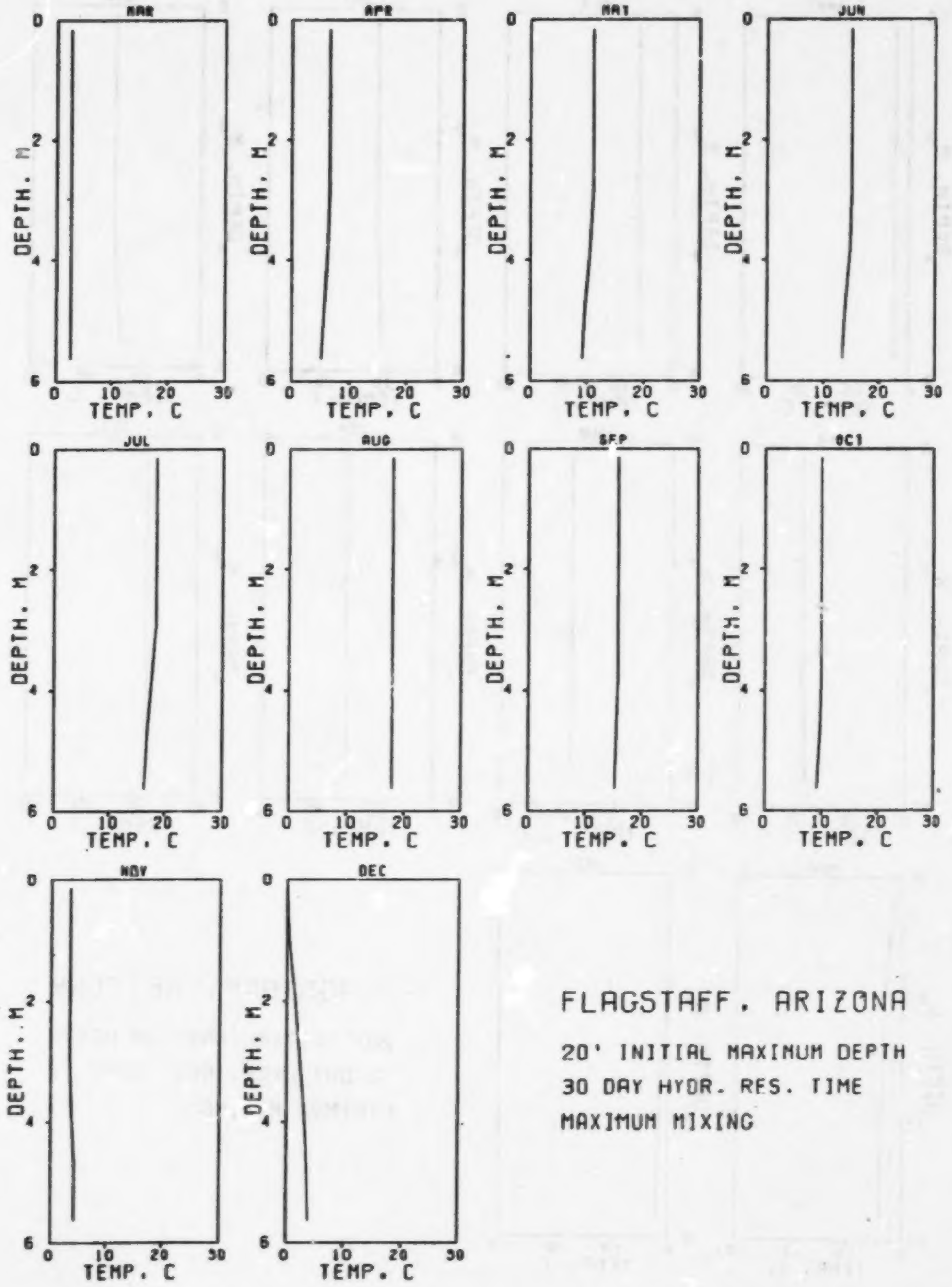
FLAGSTAFF, ARIZONA

20' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



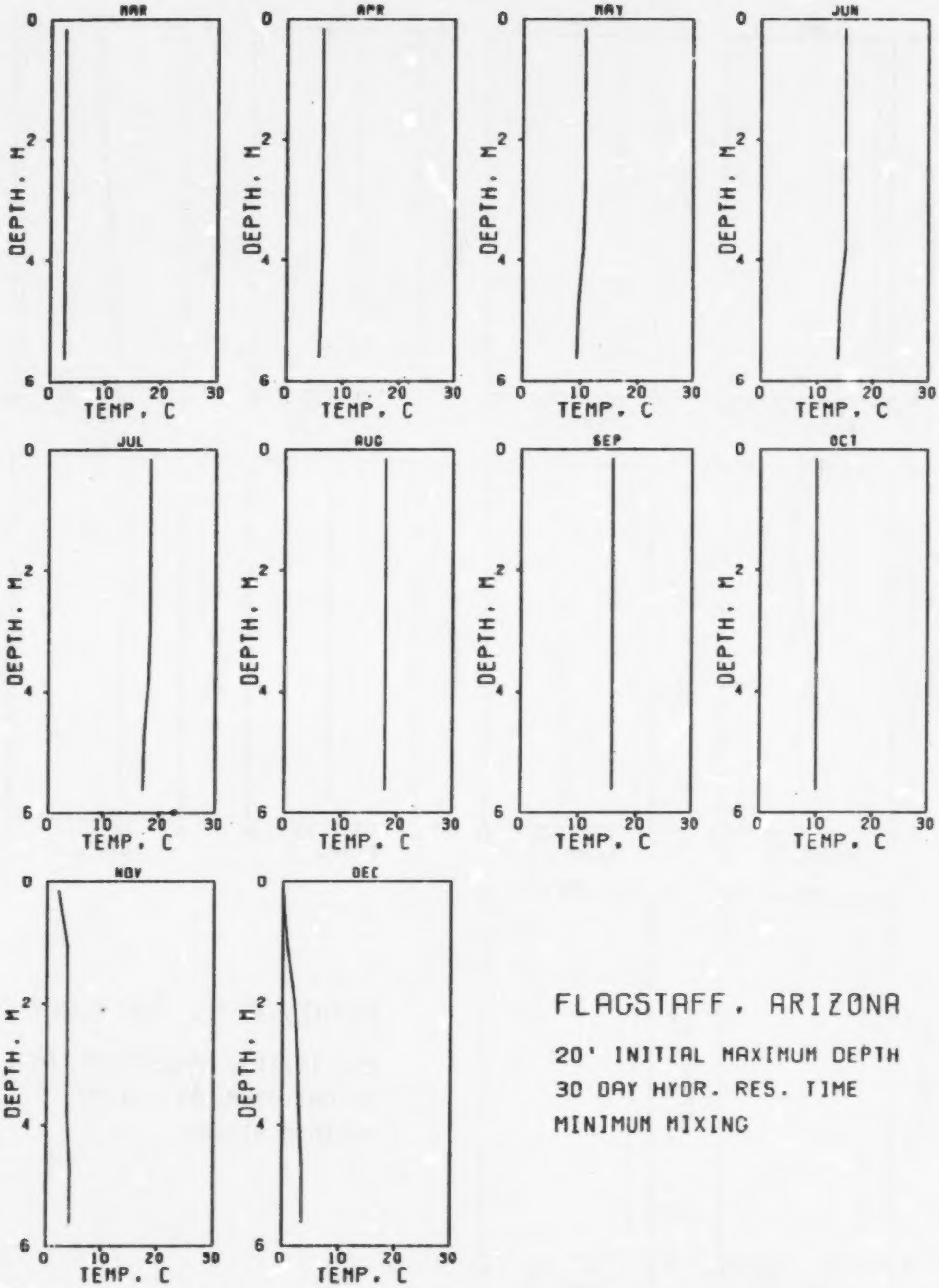
FLAGSTAFF, ARIZONA

20' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



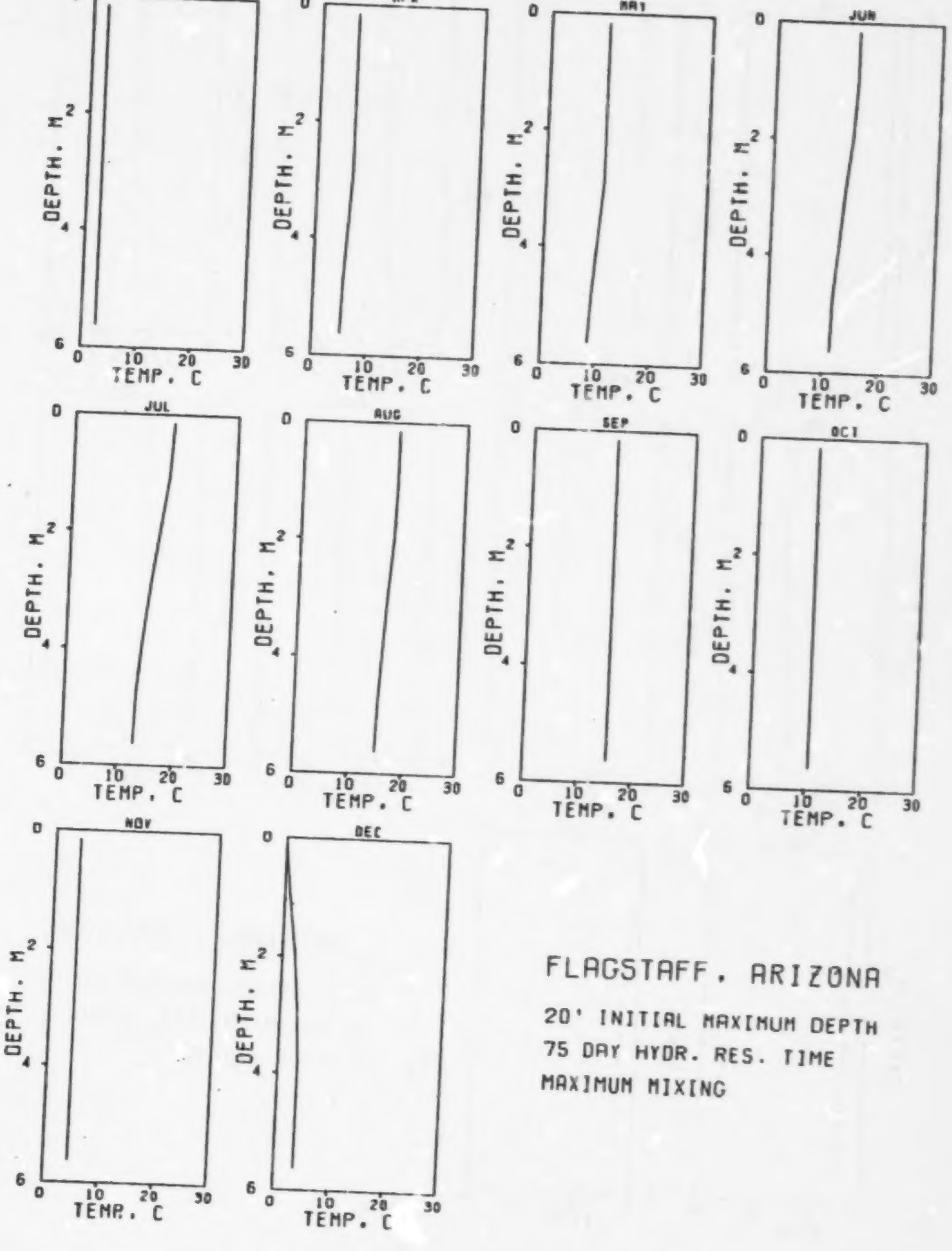
FLAGSTAFF, ARIZONA

20' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



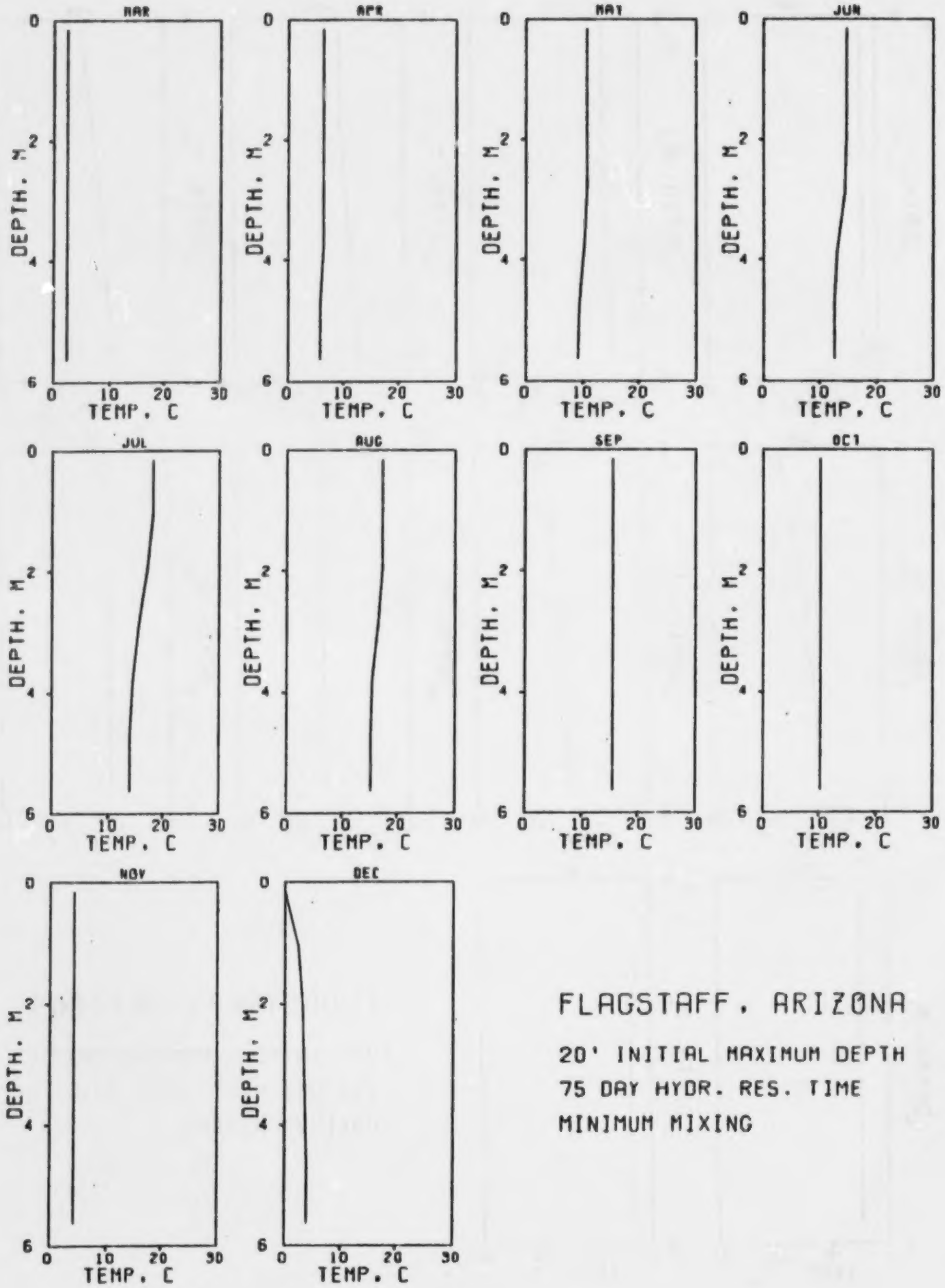
FLAGSTAFF, ARIZONA

20' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



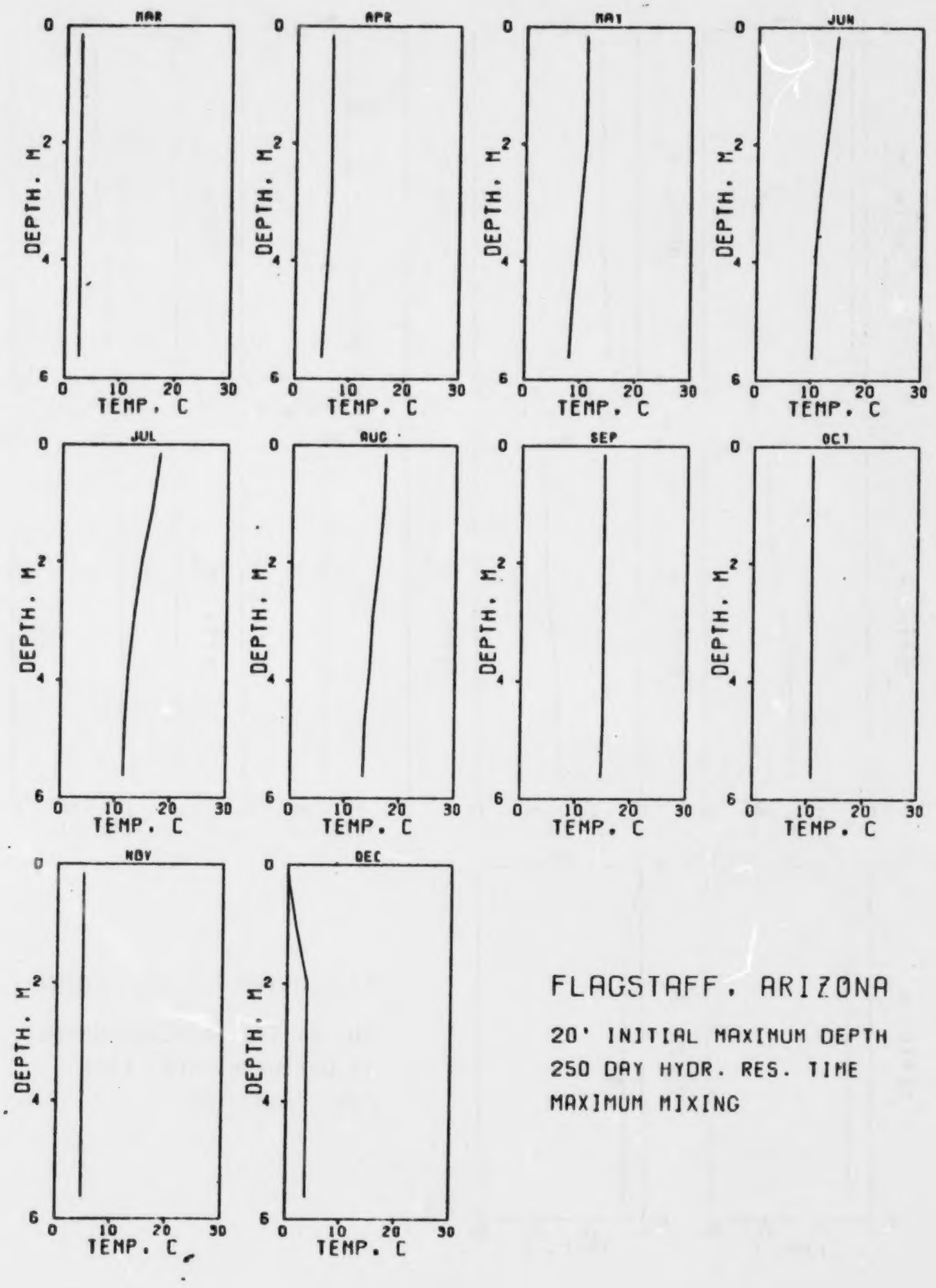
FLAGSTAFF, ARIZONA

20' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



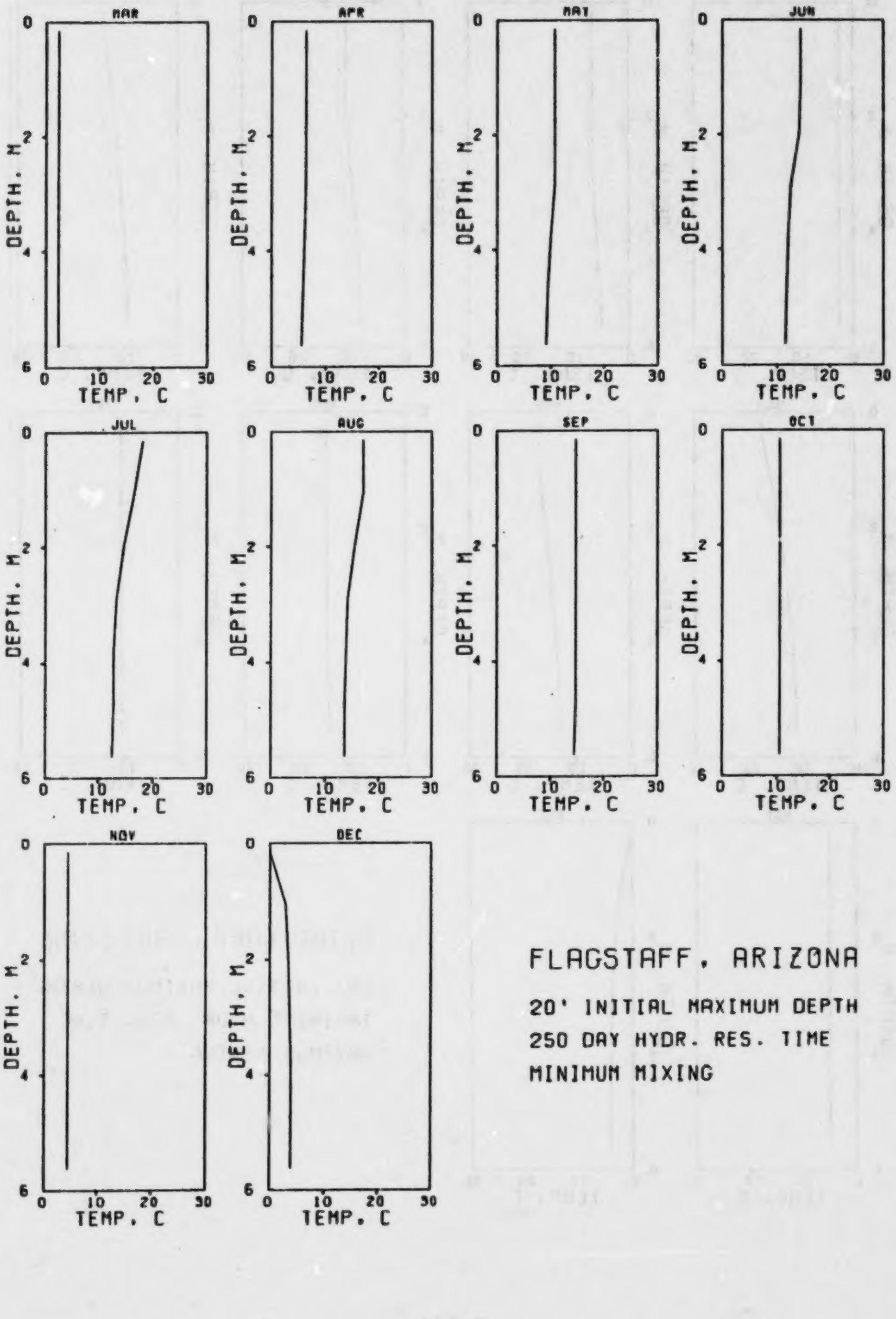
FLAGSTAFF, ARIZONA

20' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



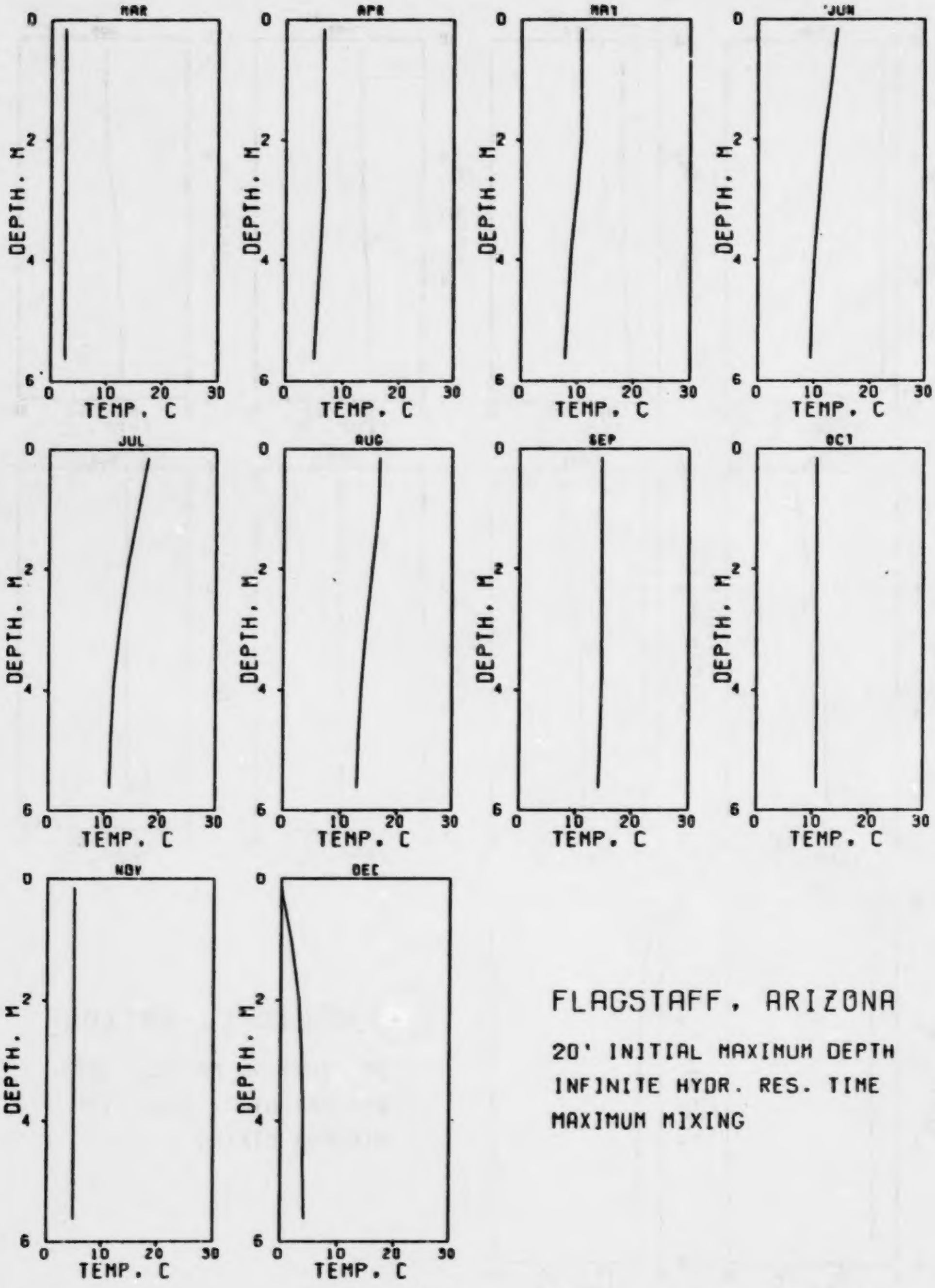
FLAGSTAFF, ARIZONA

20' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



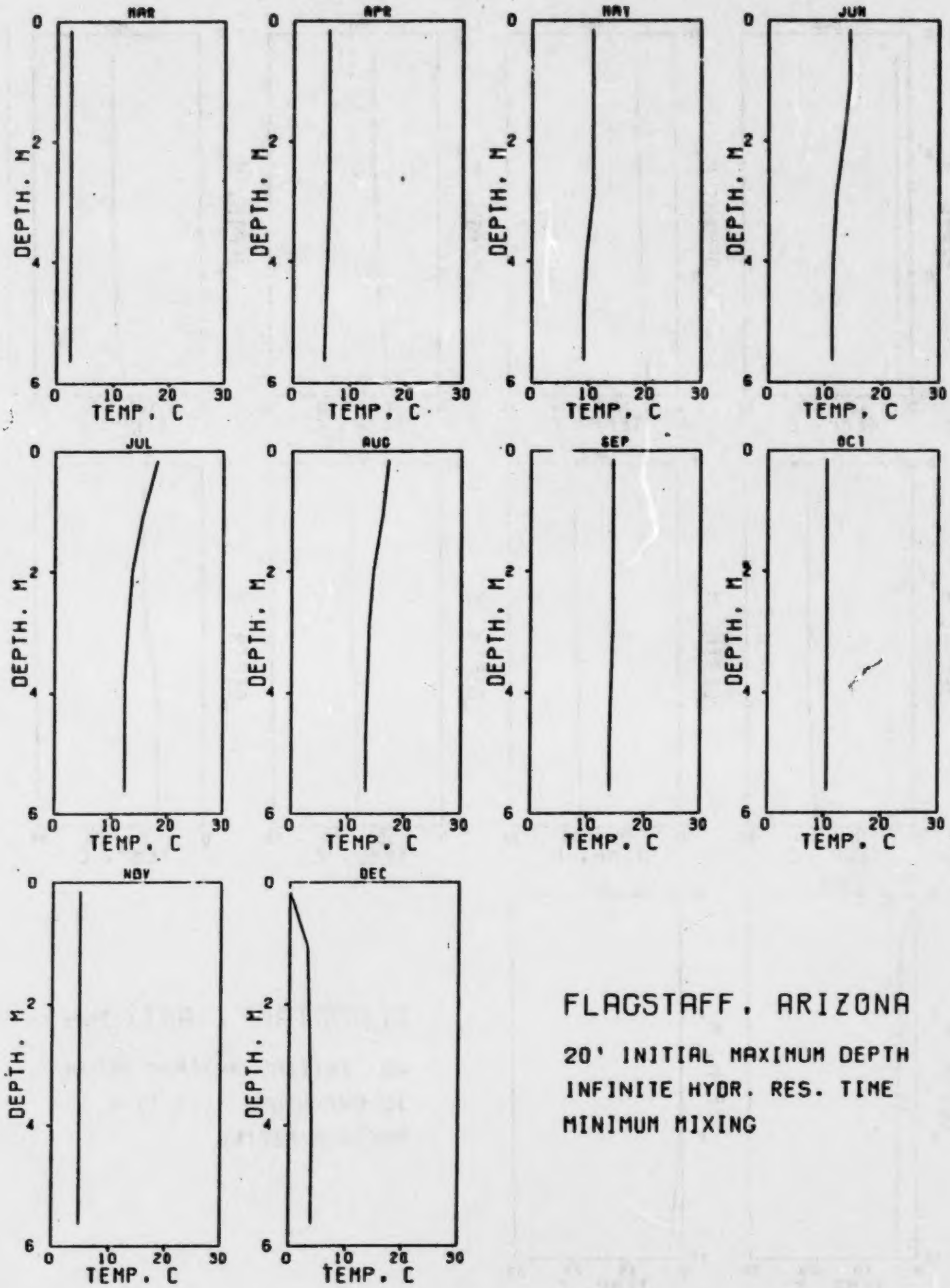
FLAGSTAFF, ARIZONA

20' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



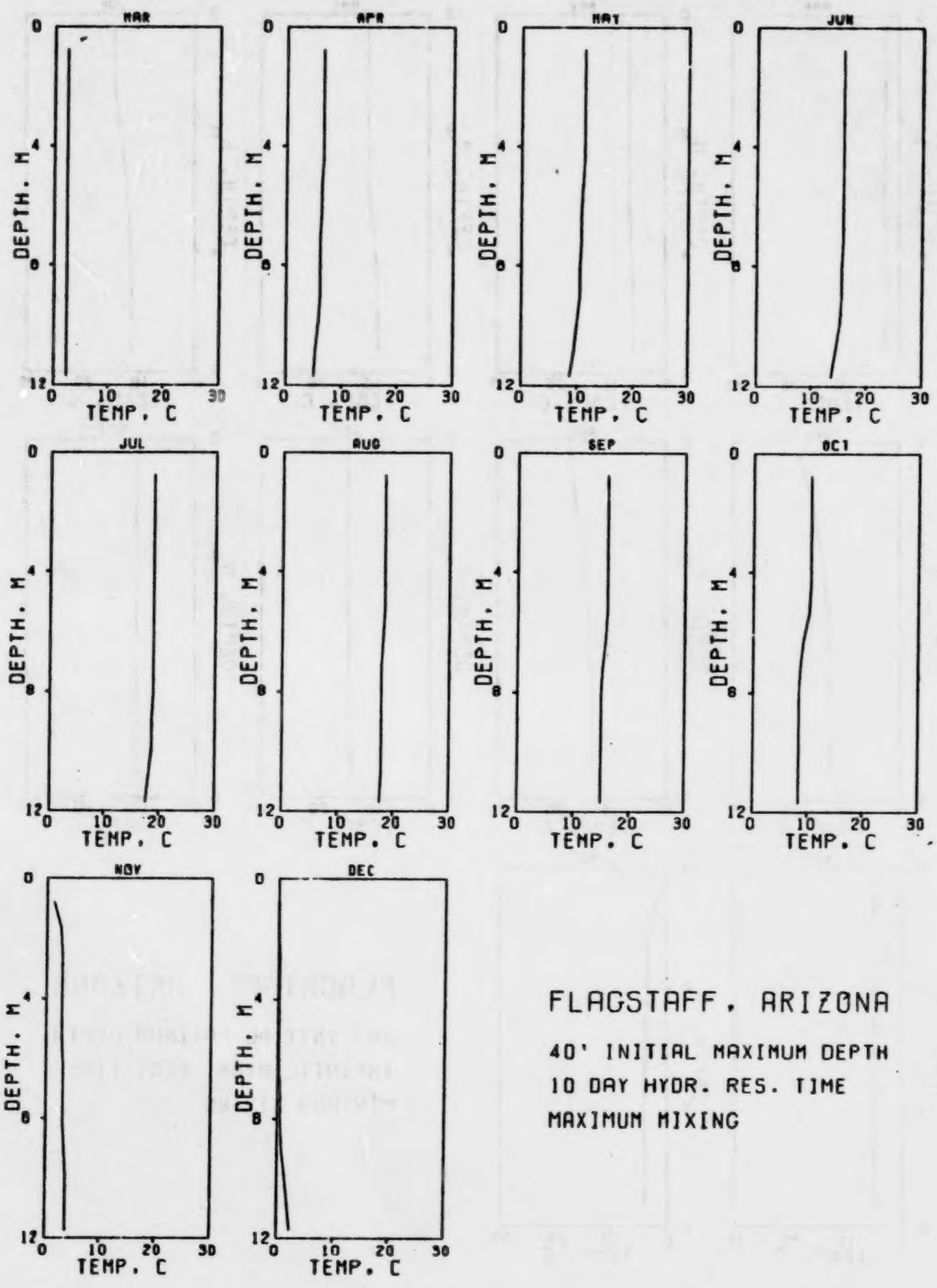
FLAGSTAFF, ARIZONA

20' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



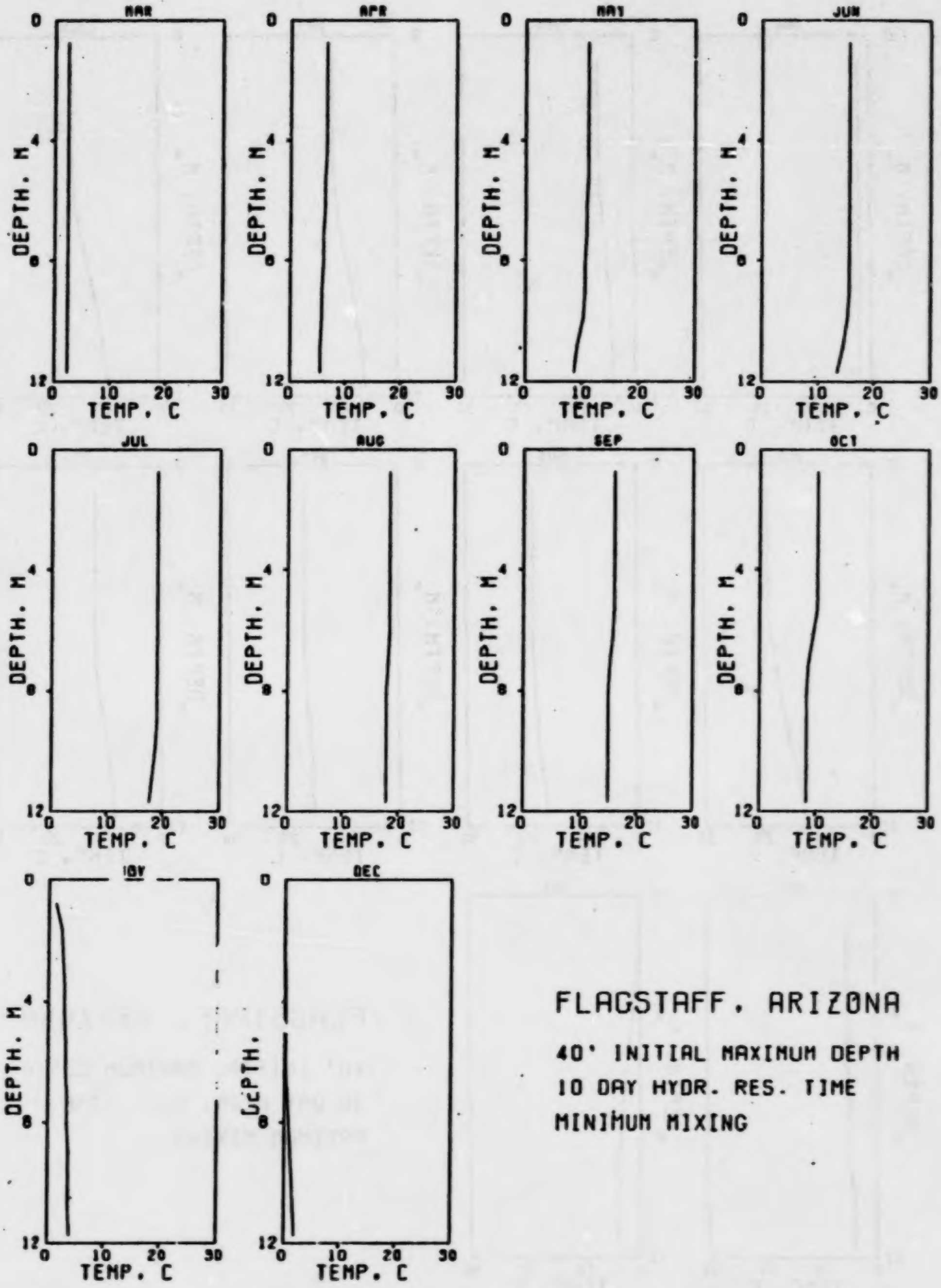
FLAGSTAFF, ARIZONA

20' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



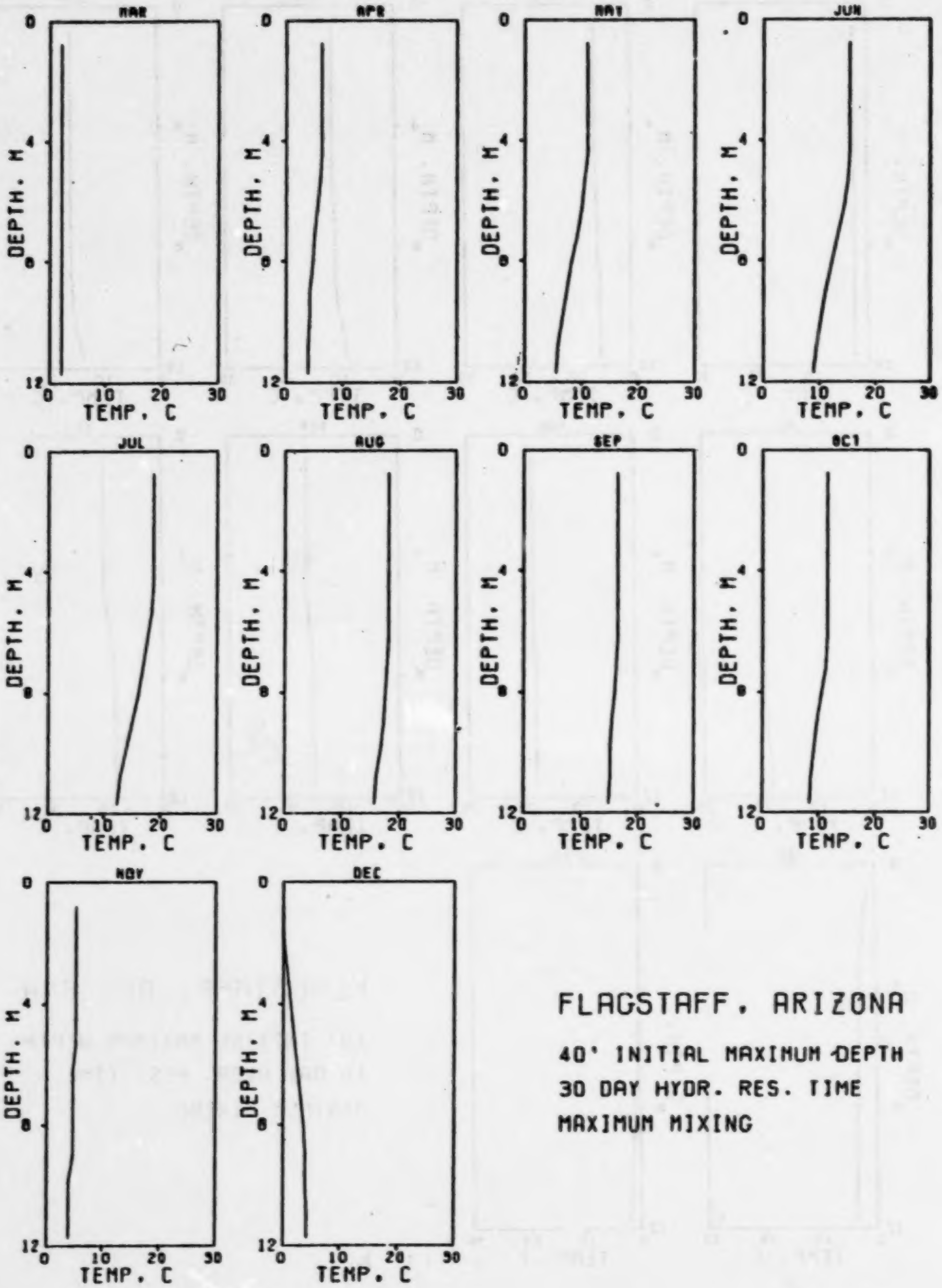
FLAGSTAFF, ARIZONA

40' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



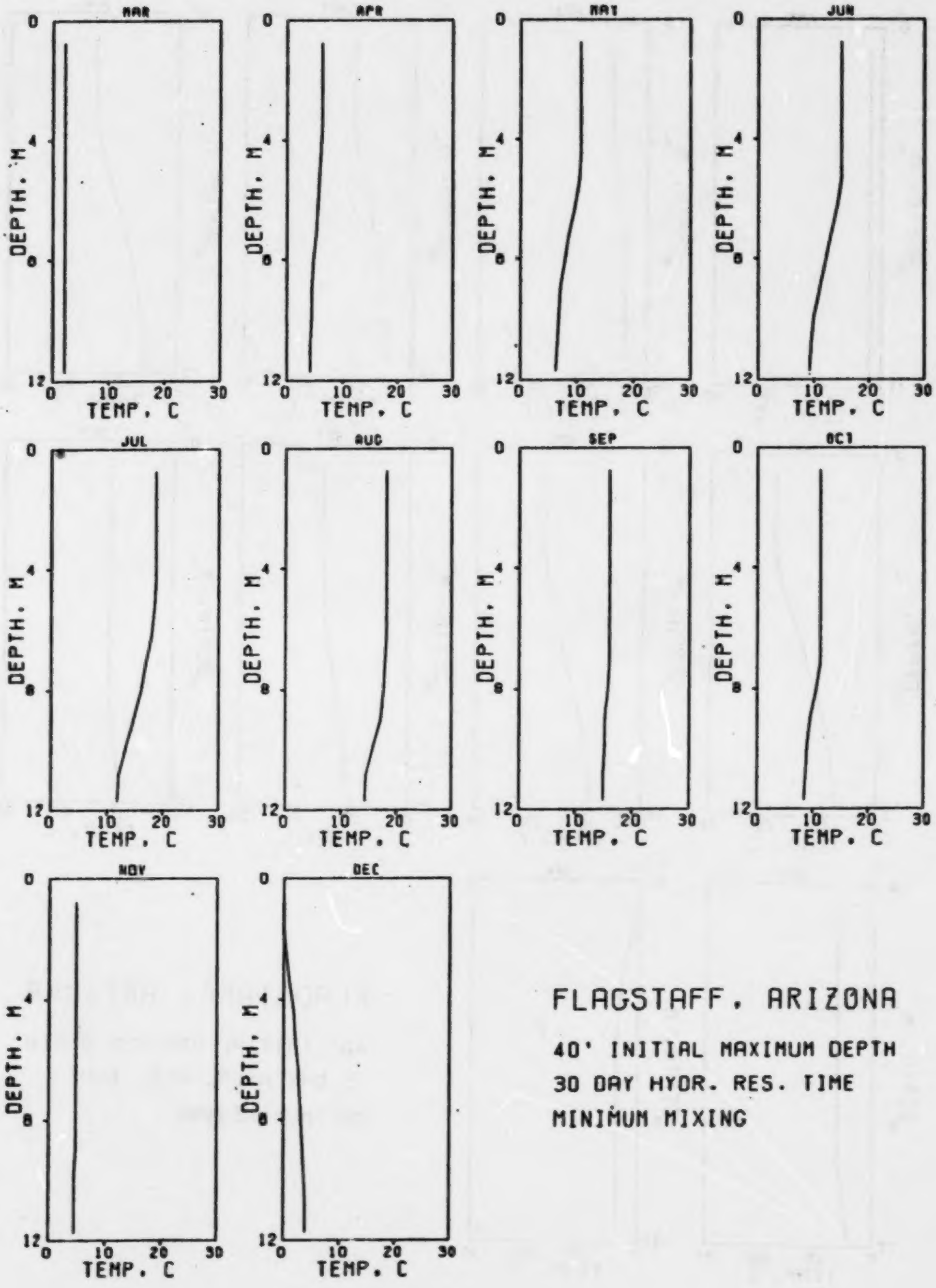
FLAGSTAFF, ARIZONA

40' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



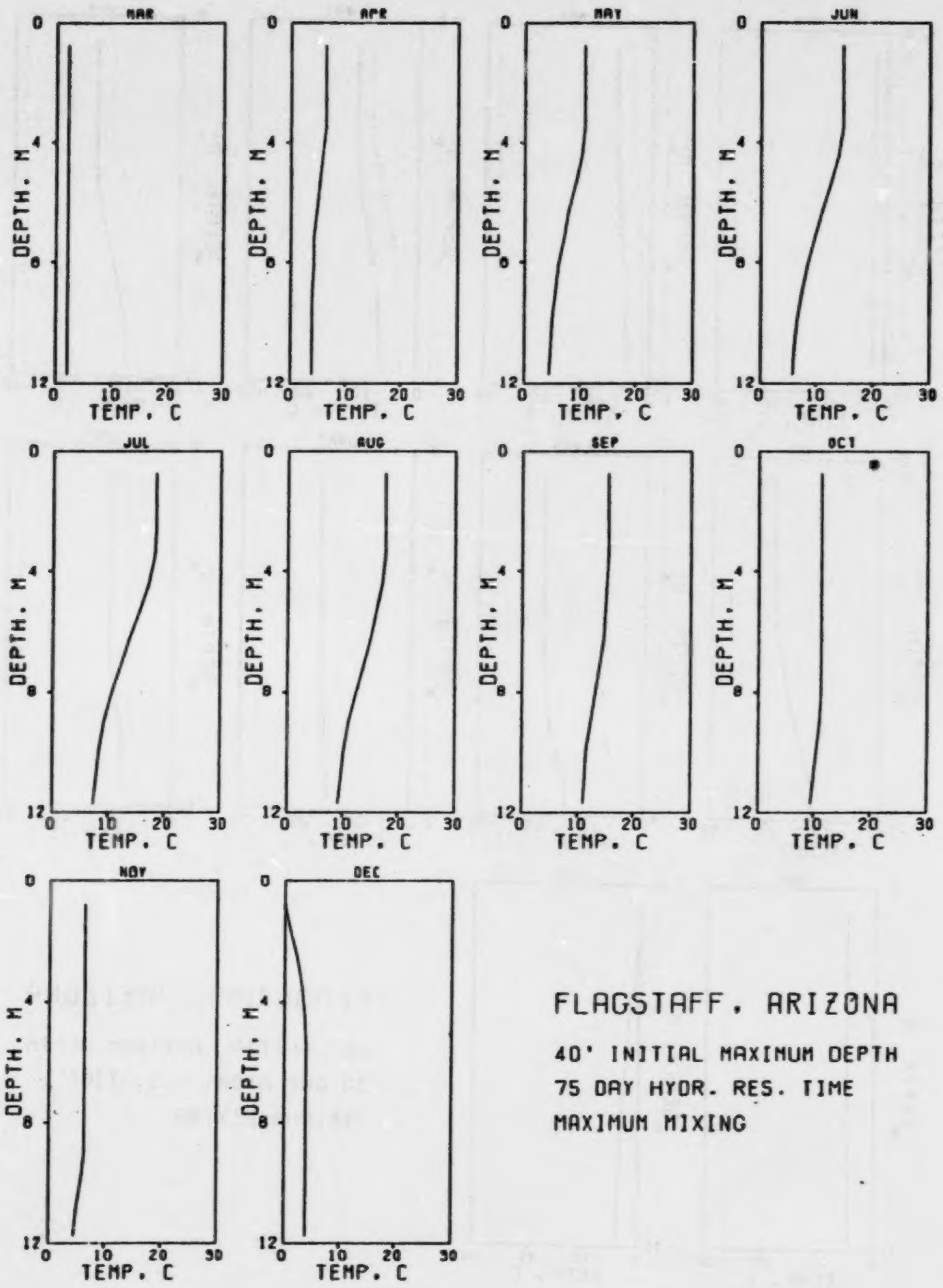
FLAGSTAFF, ARIZONA

40' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



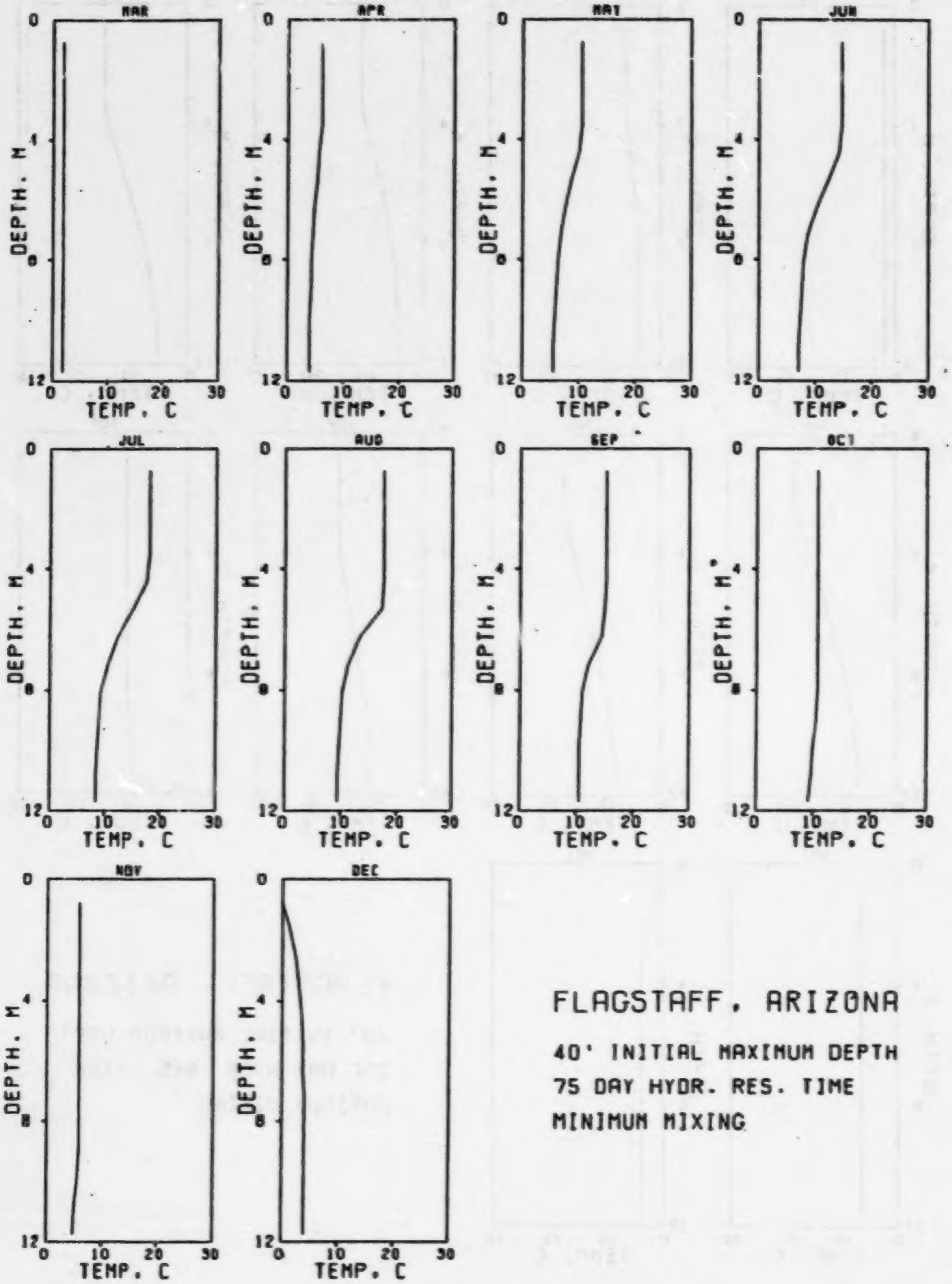
FLAGSTAFF, ARIZONA

40' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



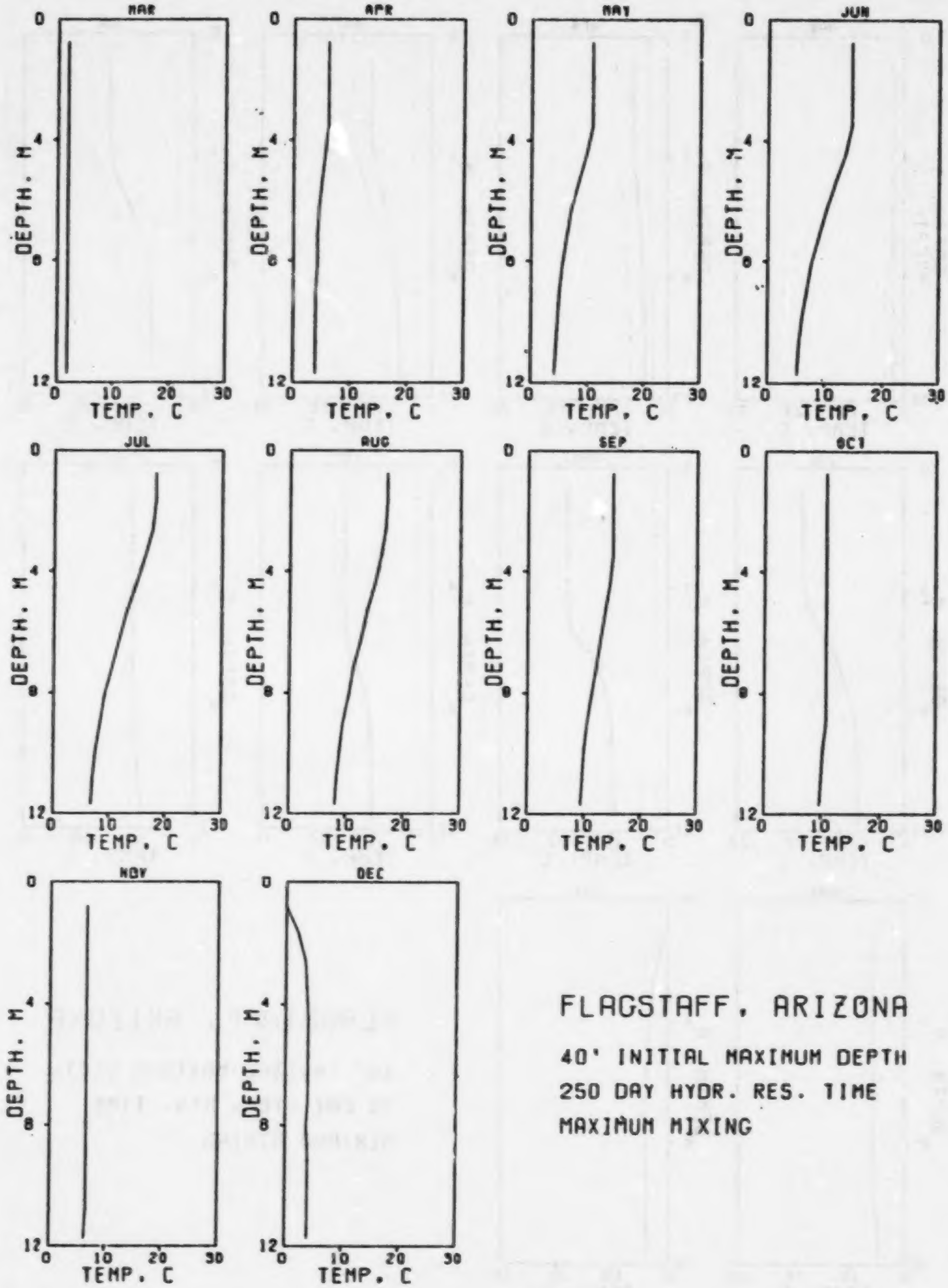
FLAGSTAFF, ARIZONA

40' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



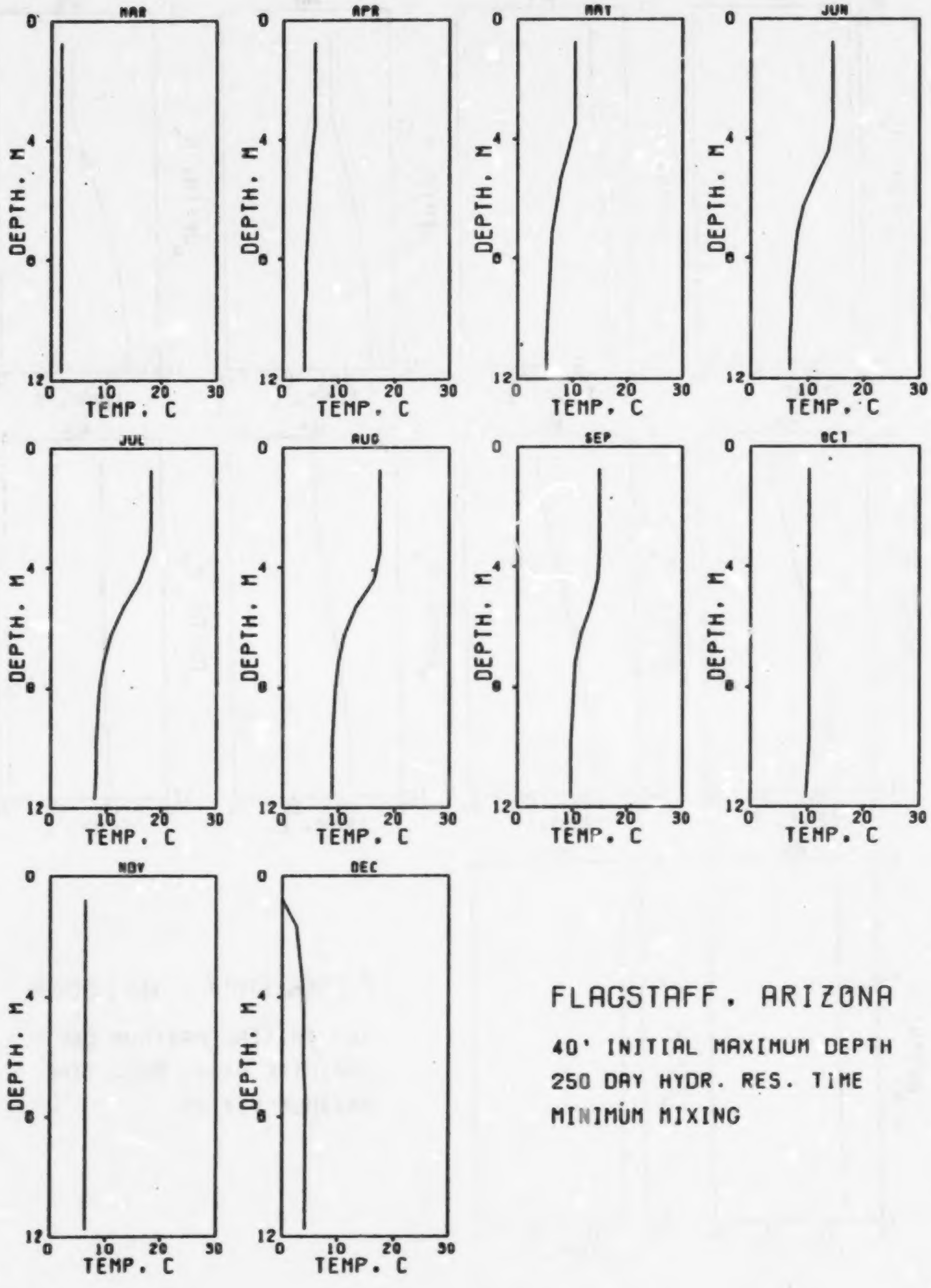
FLAGSTAFF, ARIZONA

40' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



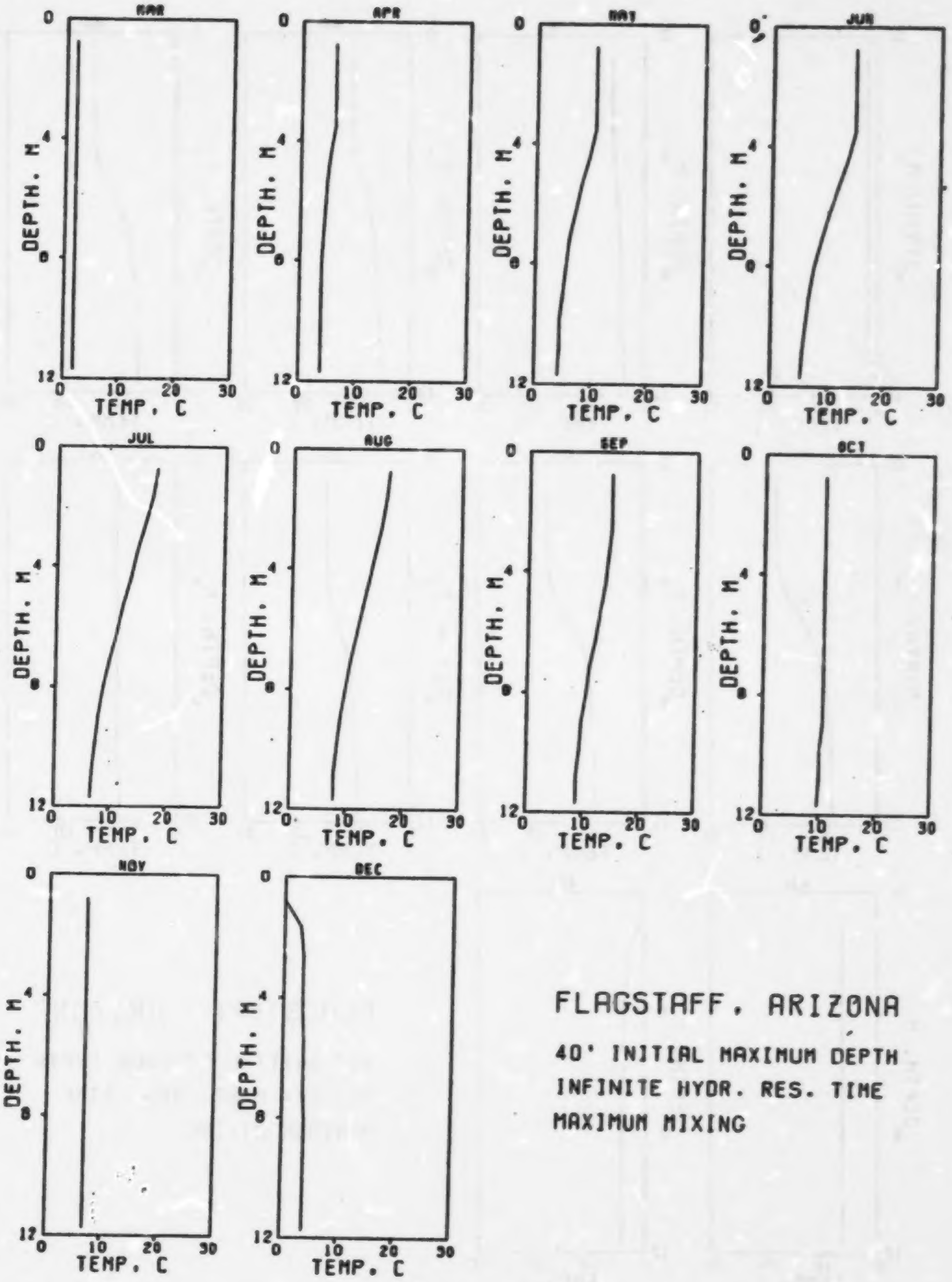
FLAGSTAFF, ARIZONA

40' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



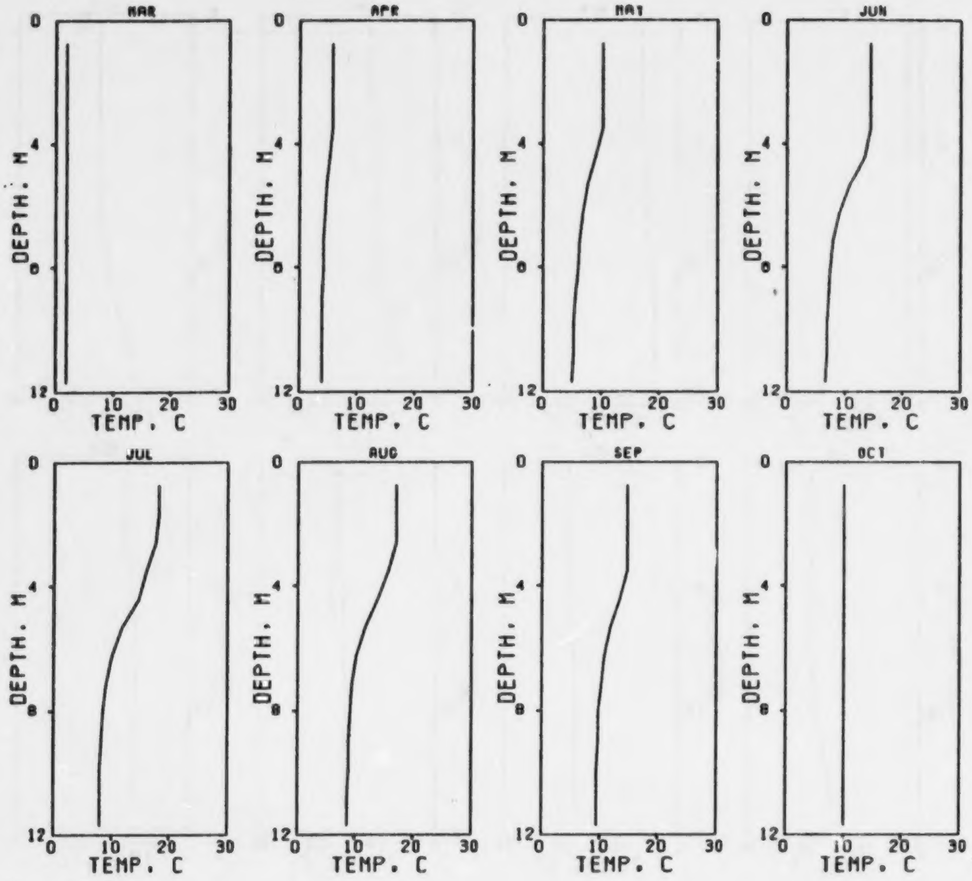
FLAGSTAFF, ARIZONA

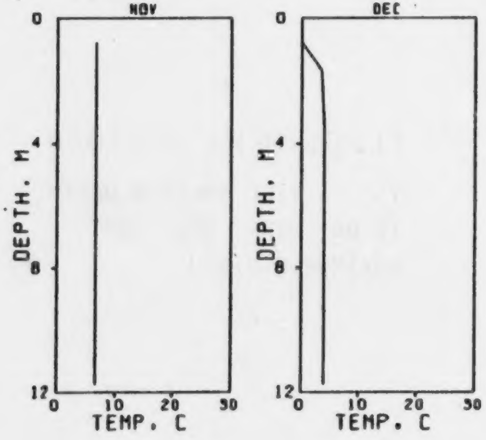
40' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



FLAGSTAFF, ARIZONA

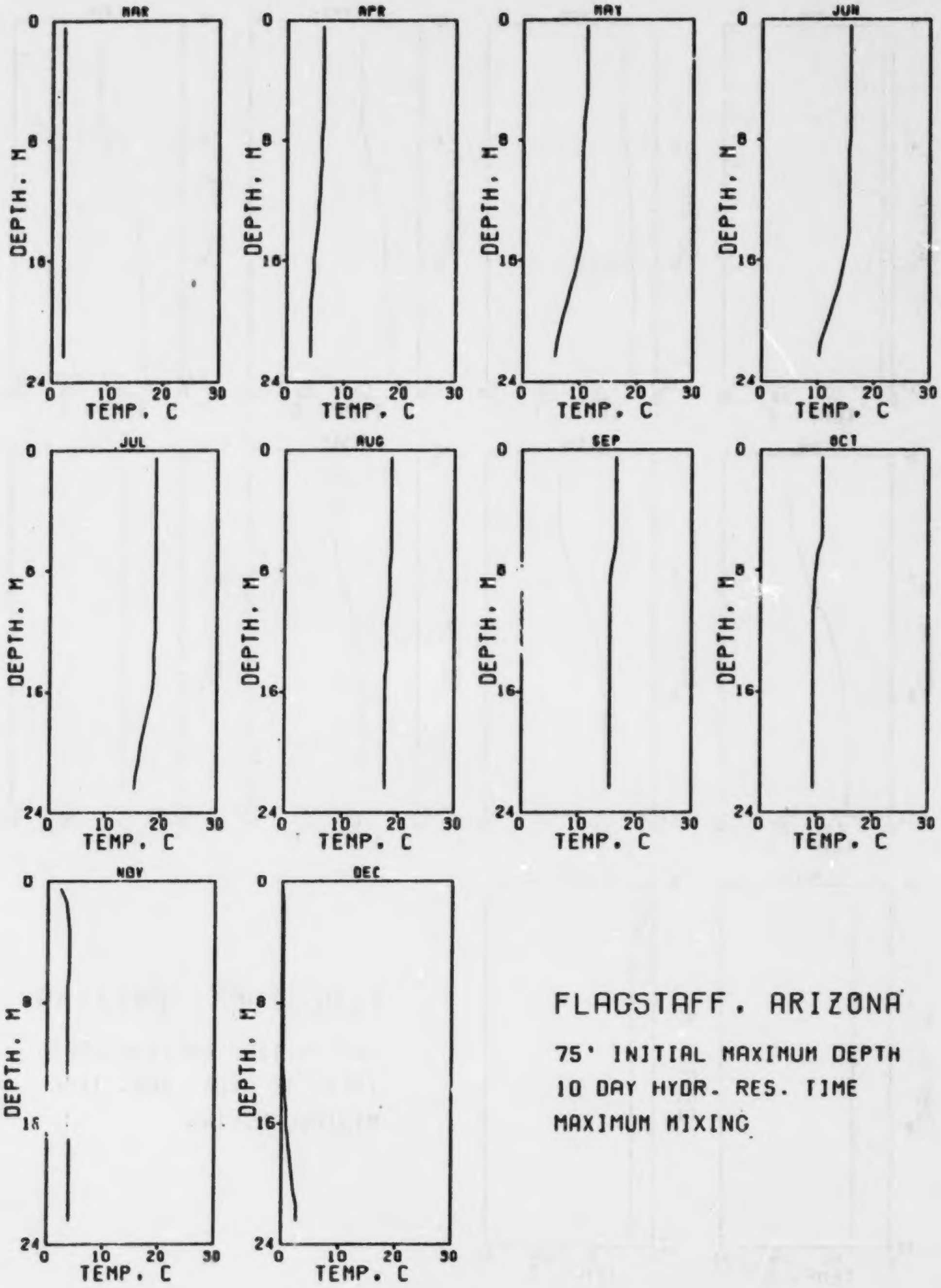
40' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING





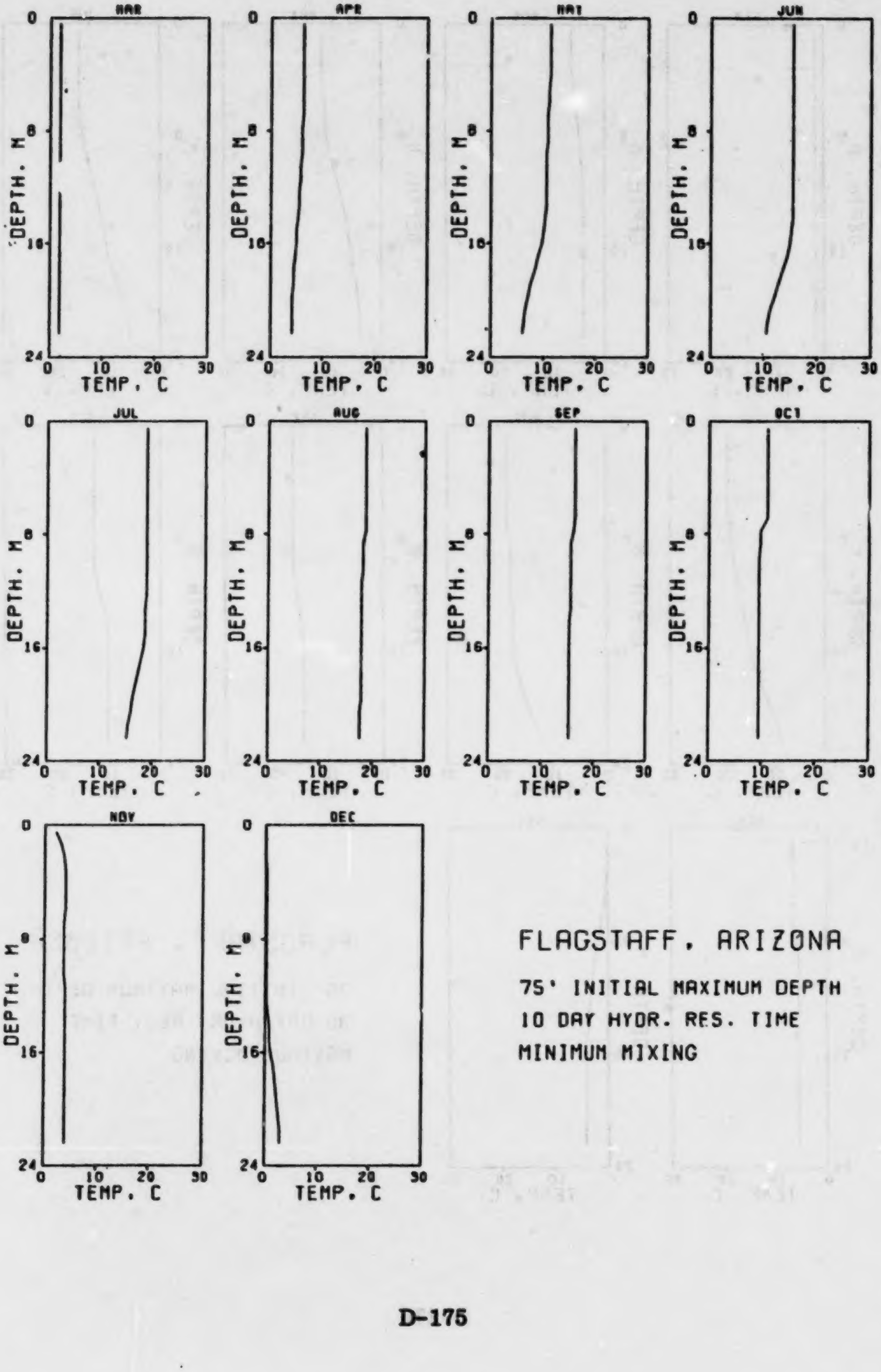
FLAGSTAFF, ARIZONA

40' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



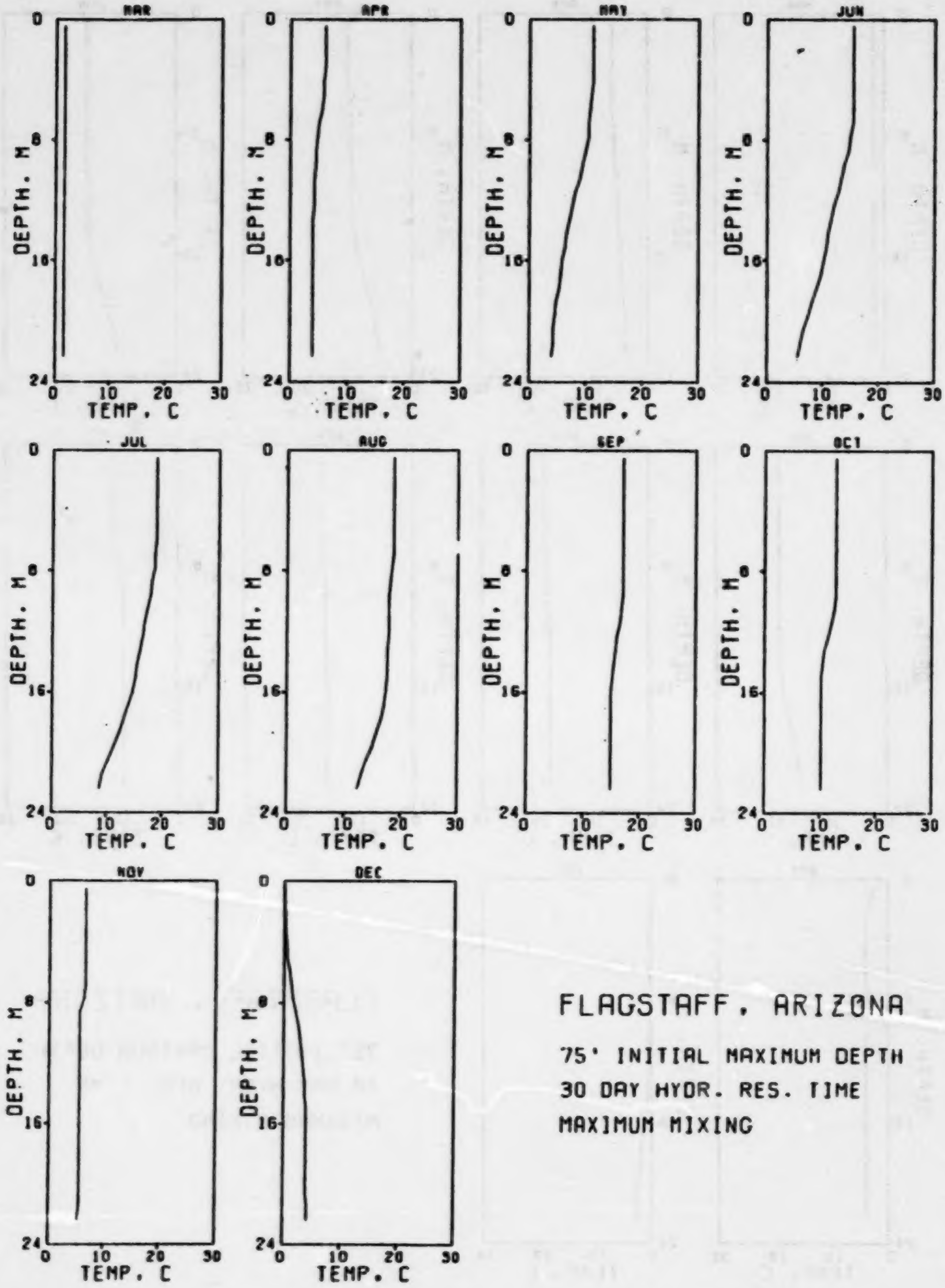
FLAGSTAFF, ARIZONA

75' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



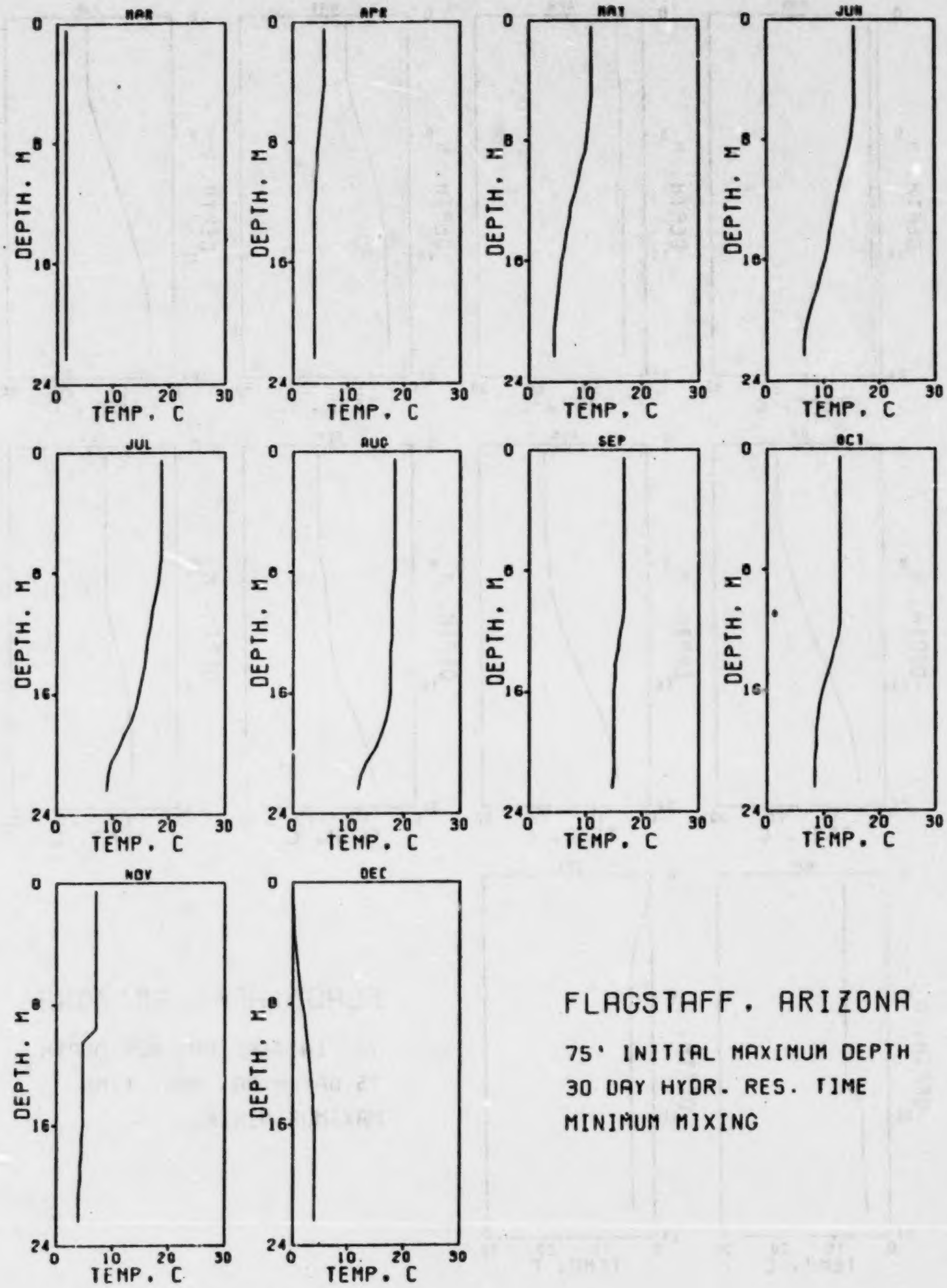
FLAGSTAFF, ARIZONA

75' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



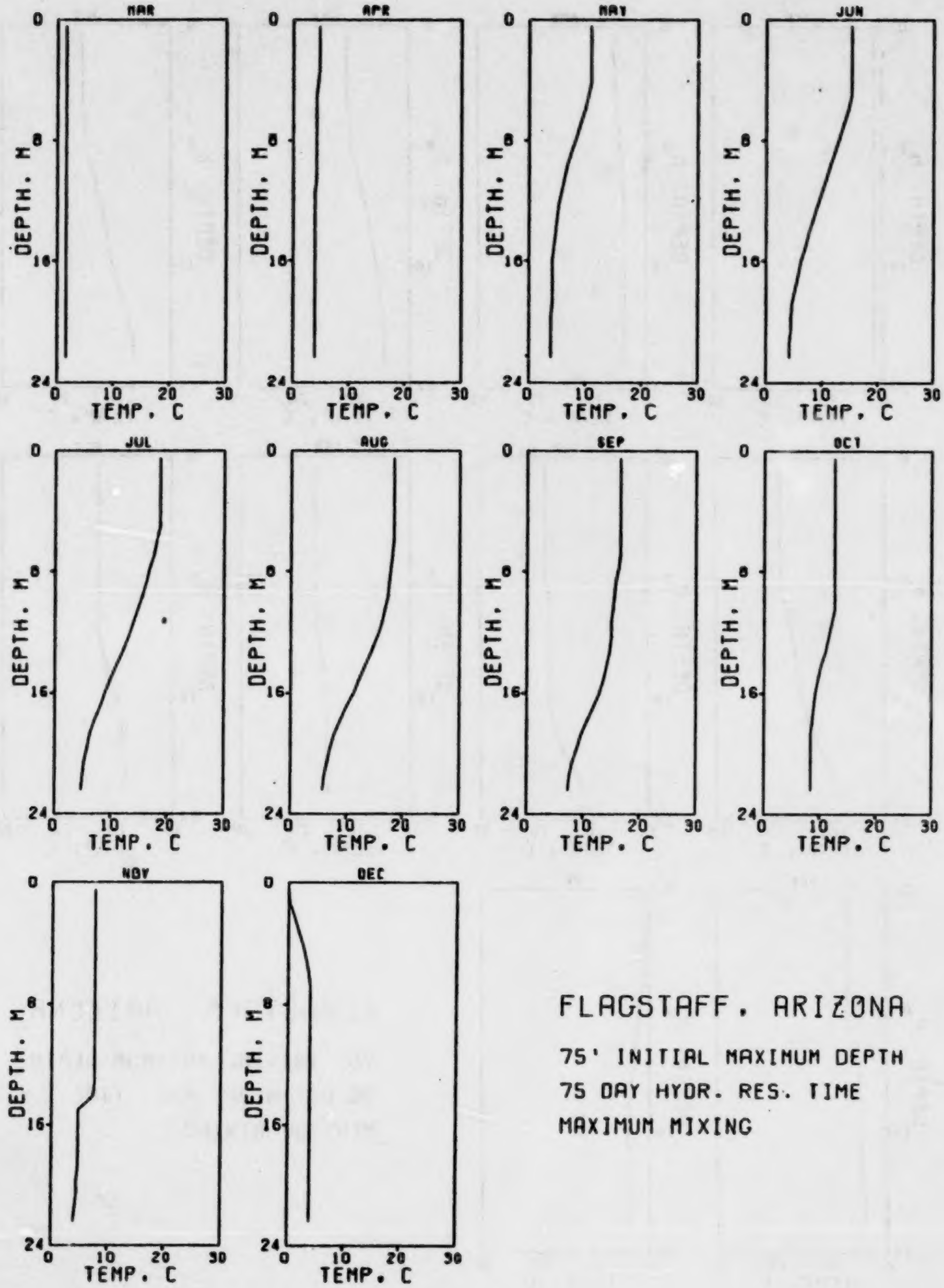
FLAGSTAFF, ARIZONA

75' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



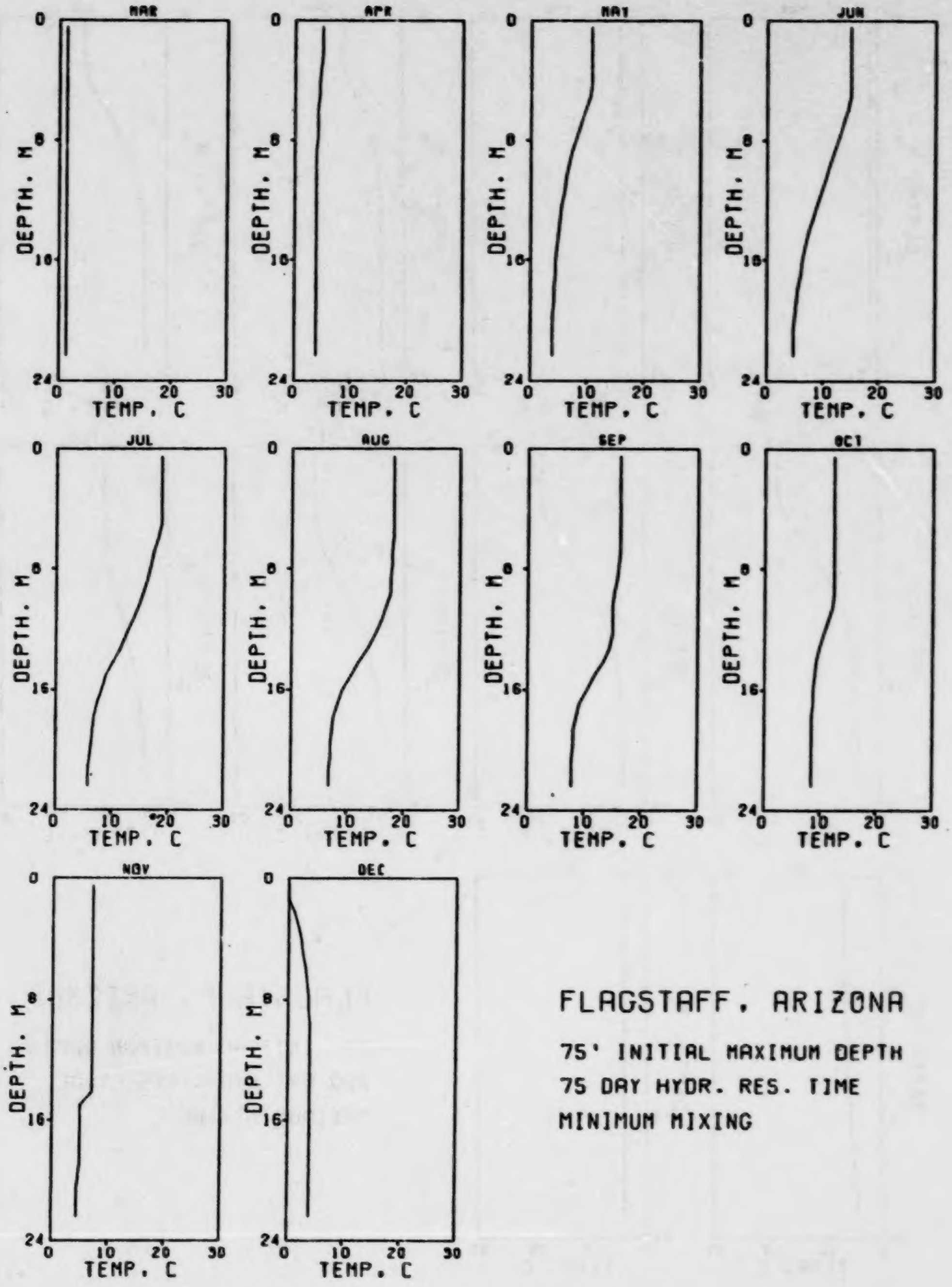
FLAGSTAFF, ARIZONA

75' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



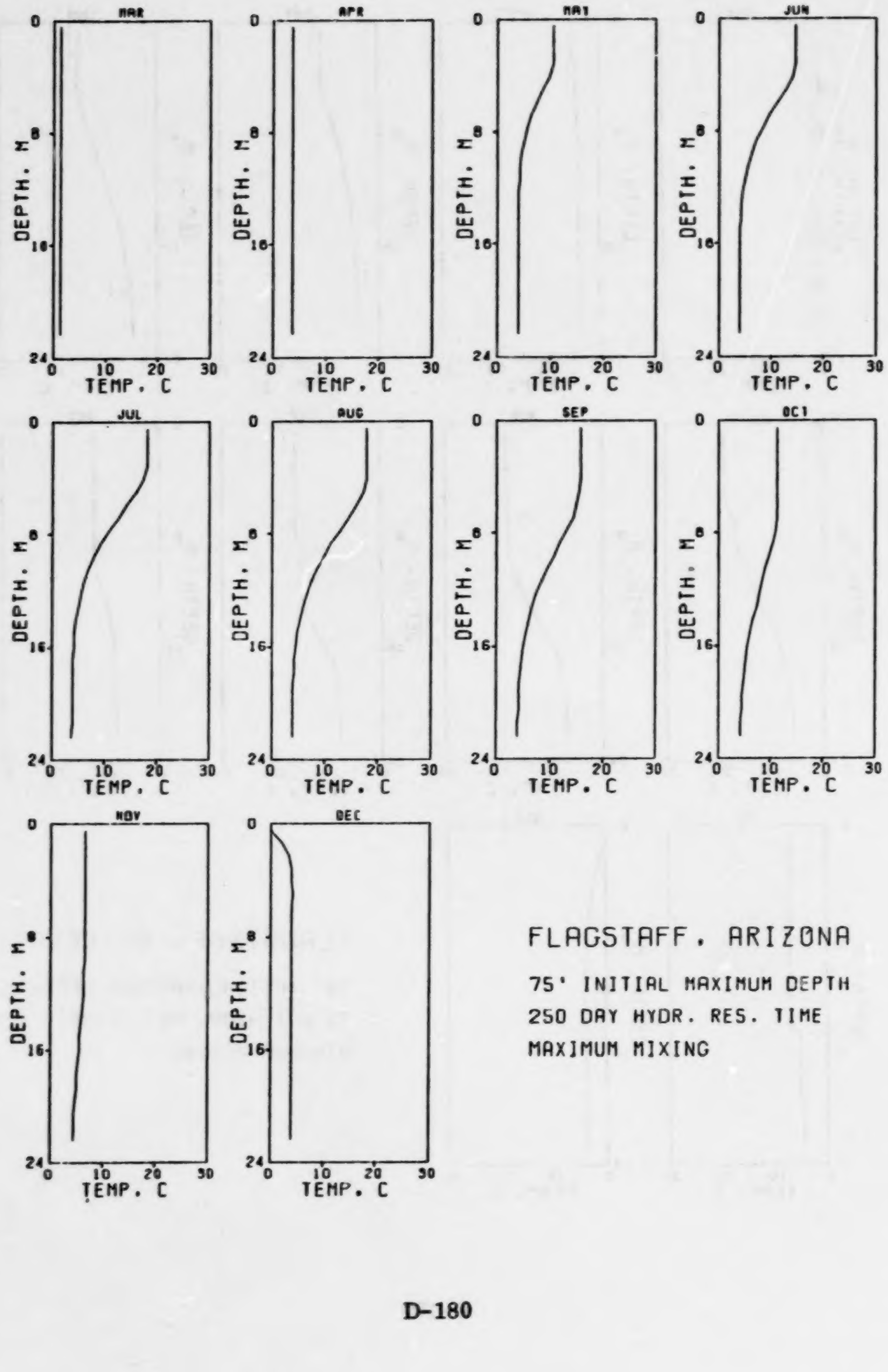
FLAGSTAFF, ARIZONA

75' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



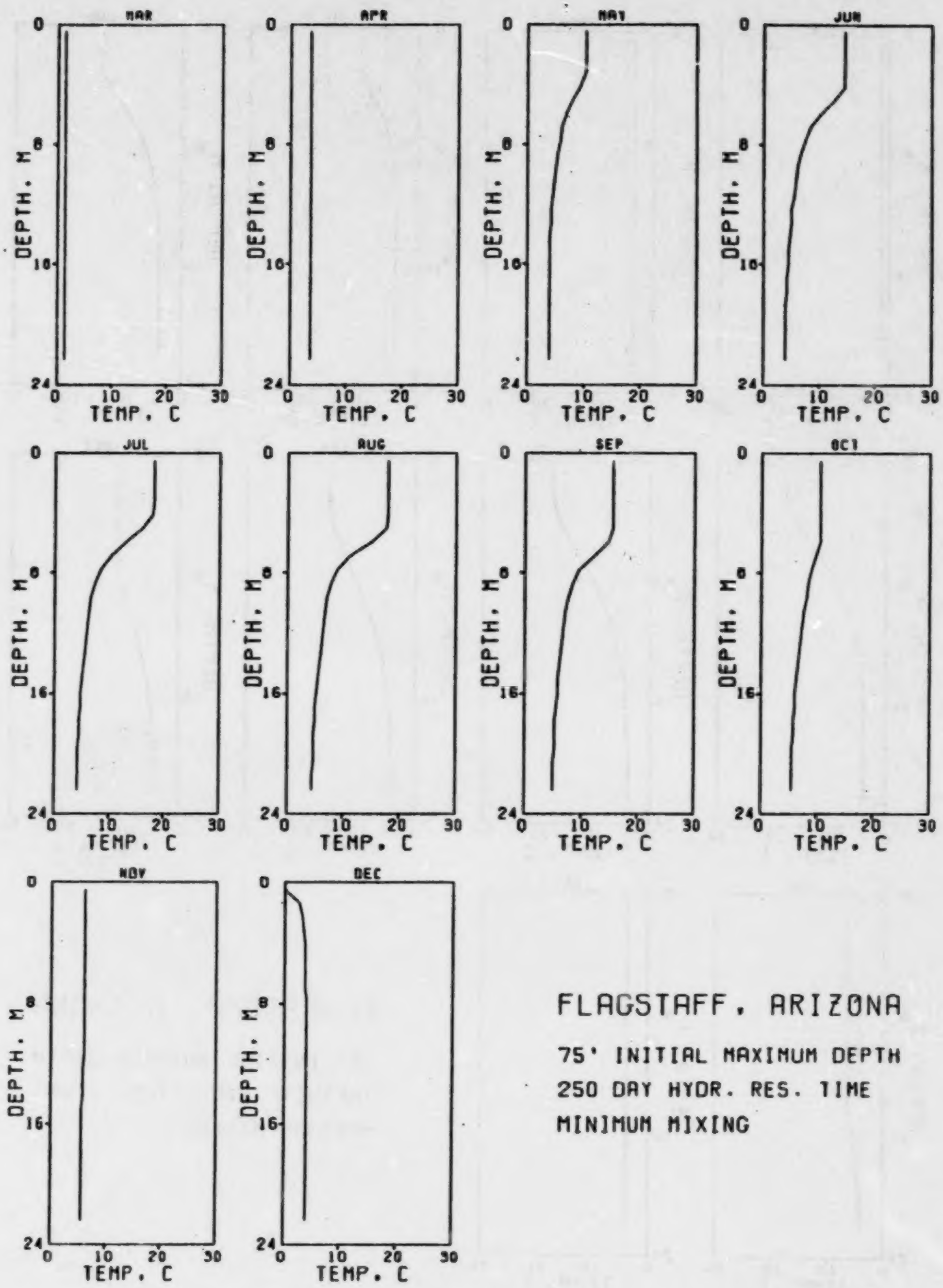
FLAGSTAFF, ARIZONA

75' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



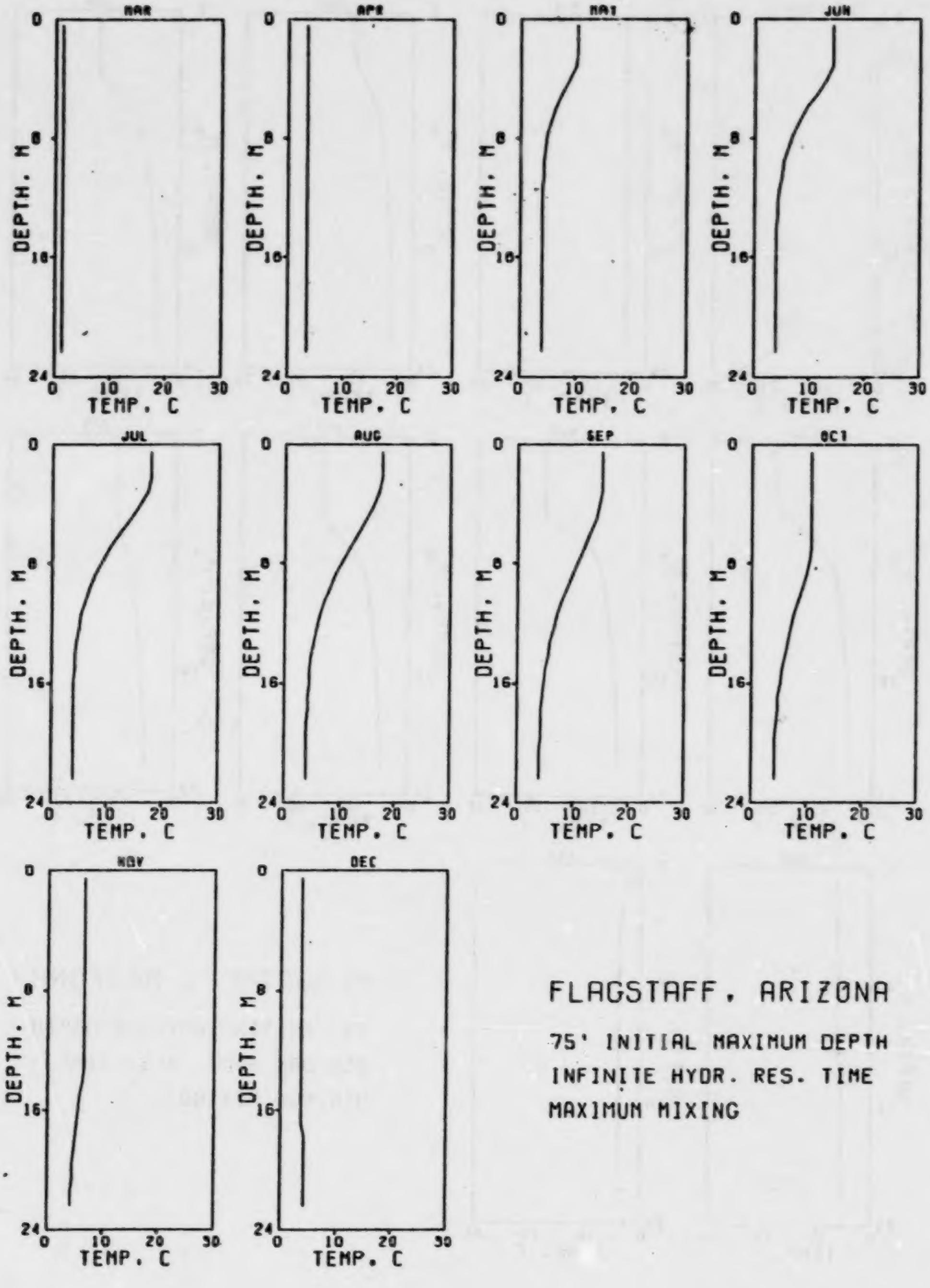
FLAGSTAFF, ARIZONA

75' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



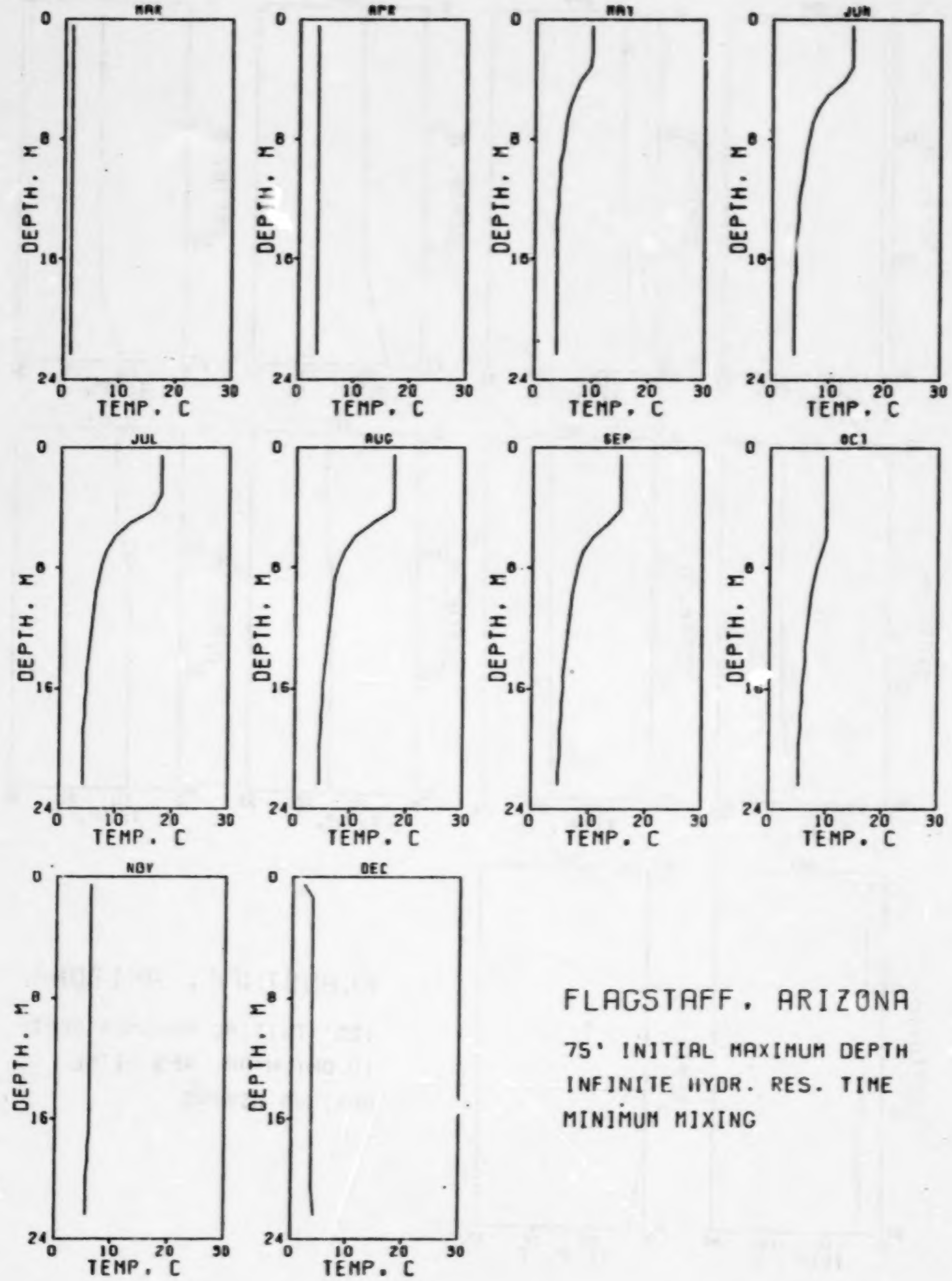
FLAGSTAFF, ARIZONA

75' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



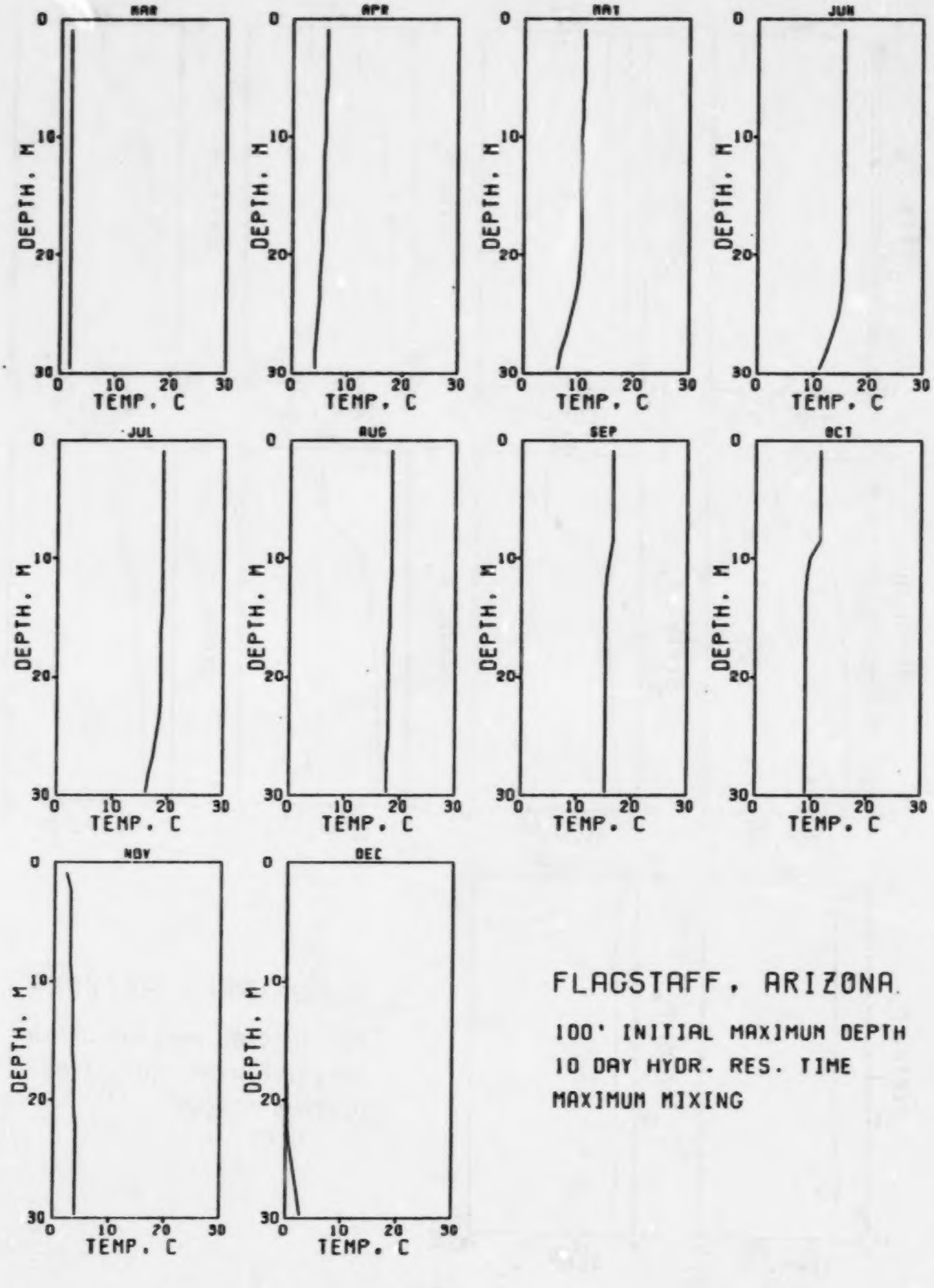
FLAGSTAFF, ARIZONA

75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



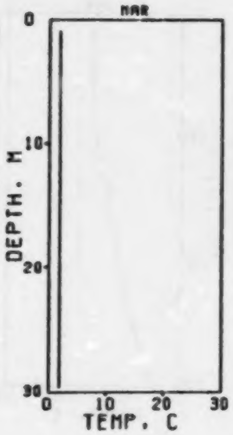
FLAGSTAFF, ARIZONA

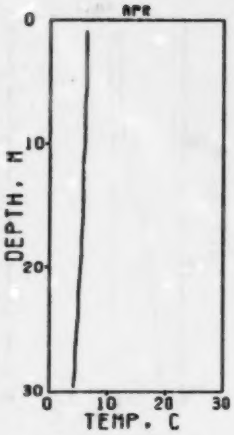
75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING

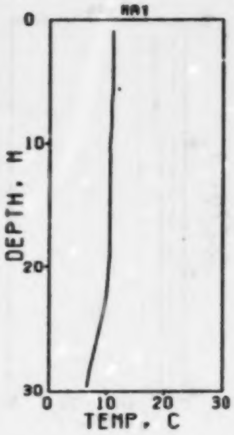


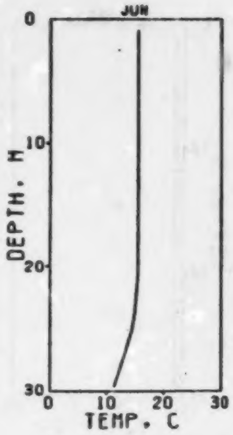
FLAGSTAFF, ARIZONA

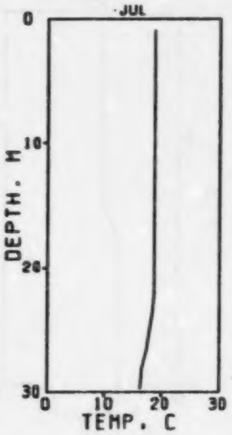
100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING

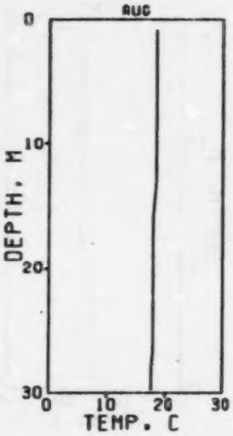


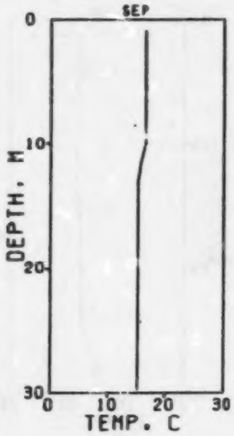


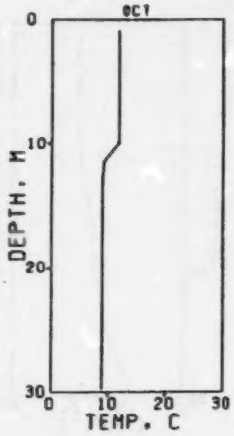


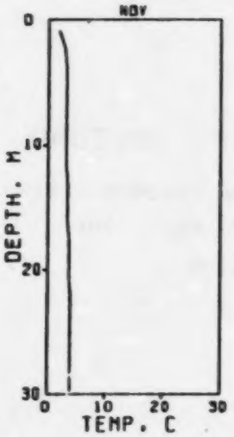


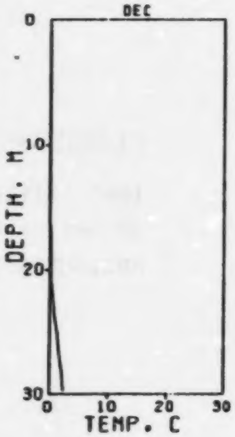






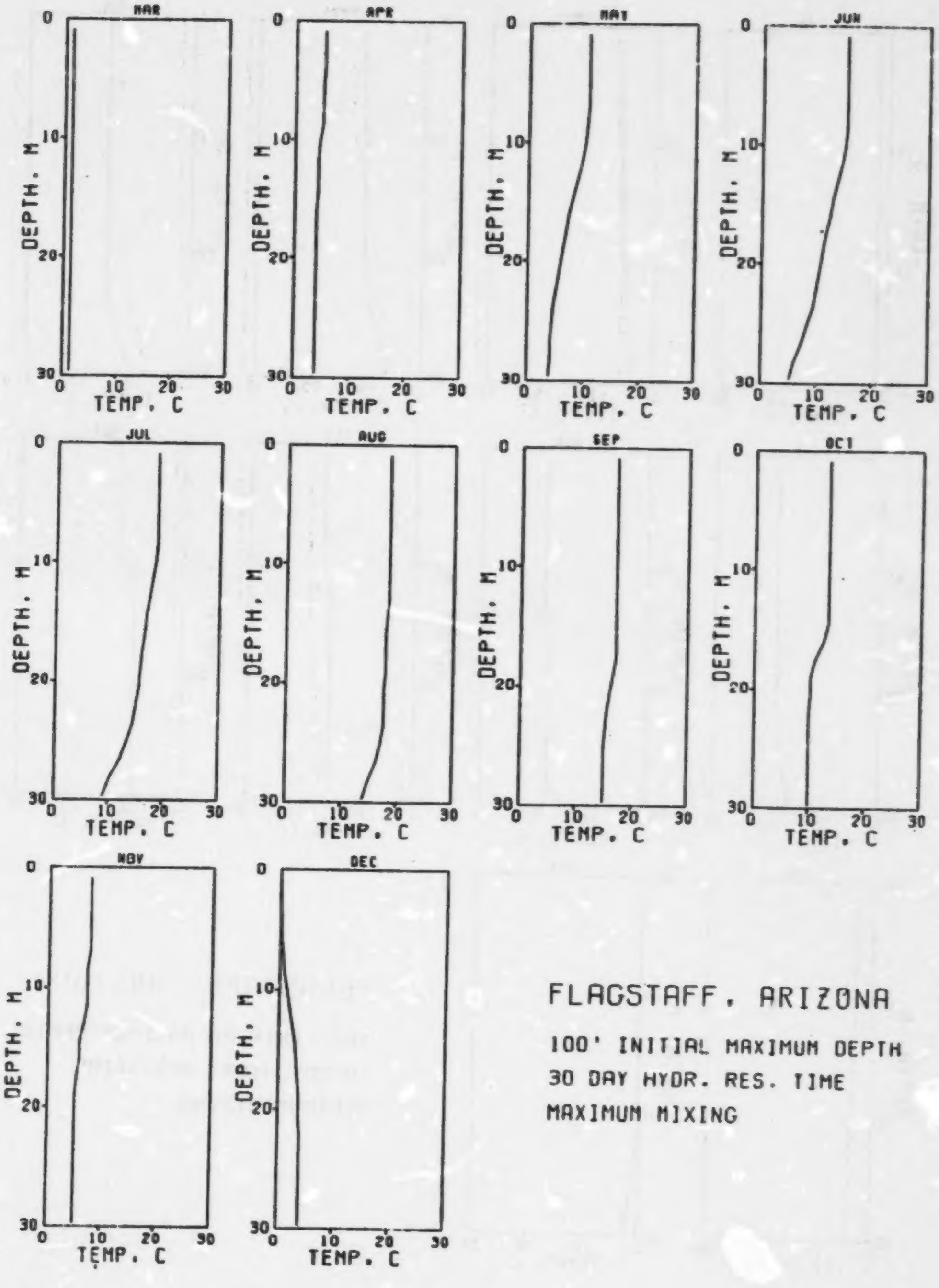






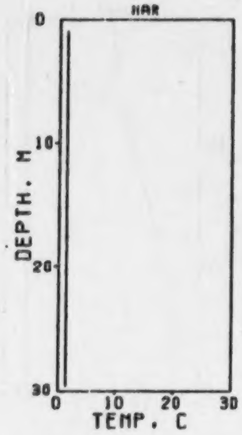
FLAGSTAFF, ARIZONA

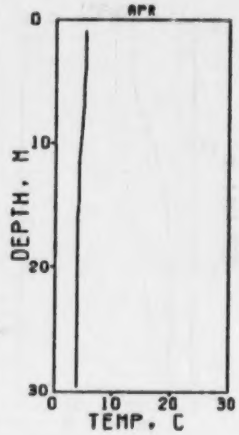
100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING

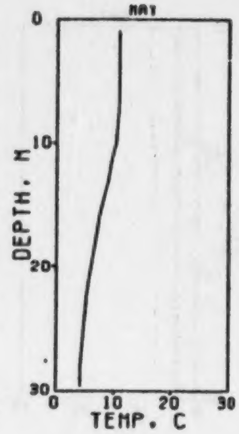


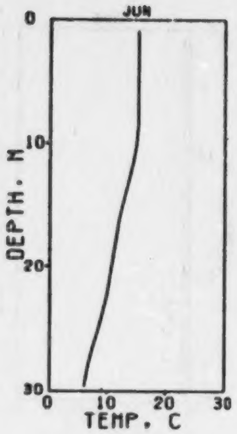
FLAGSTAFF, ARIZONA

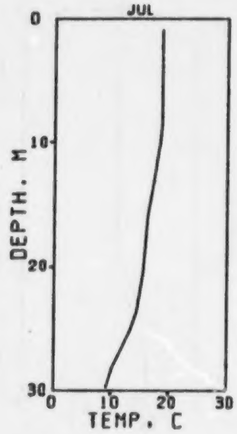
100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING

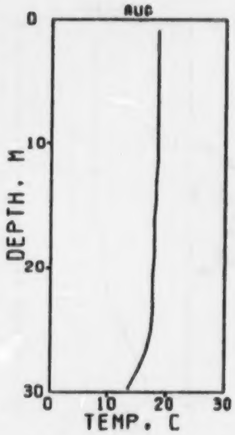


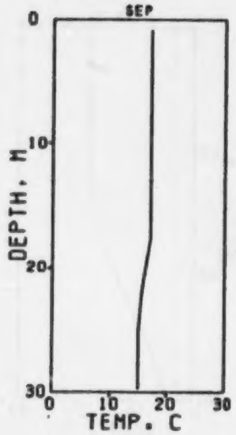


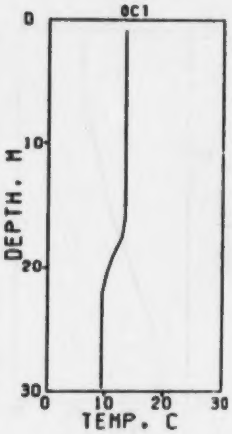


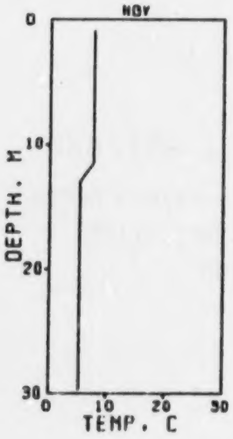


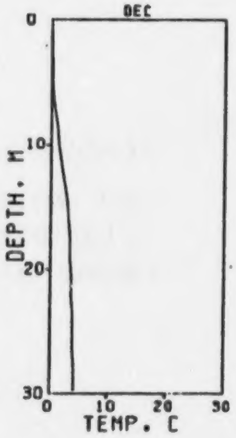






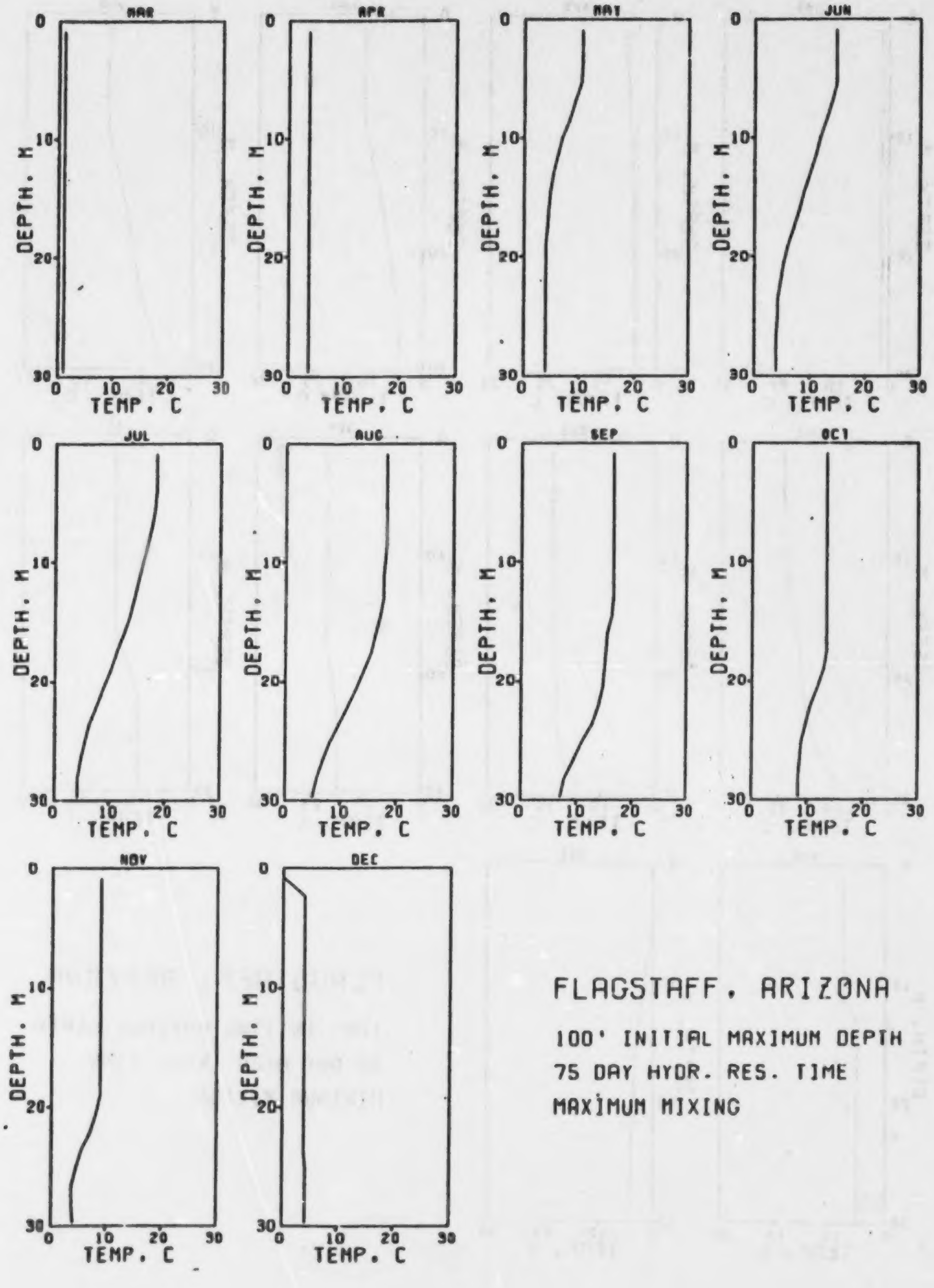






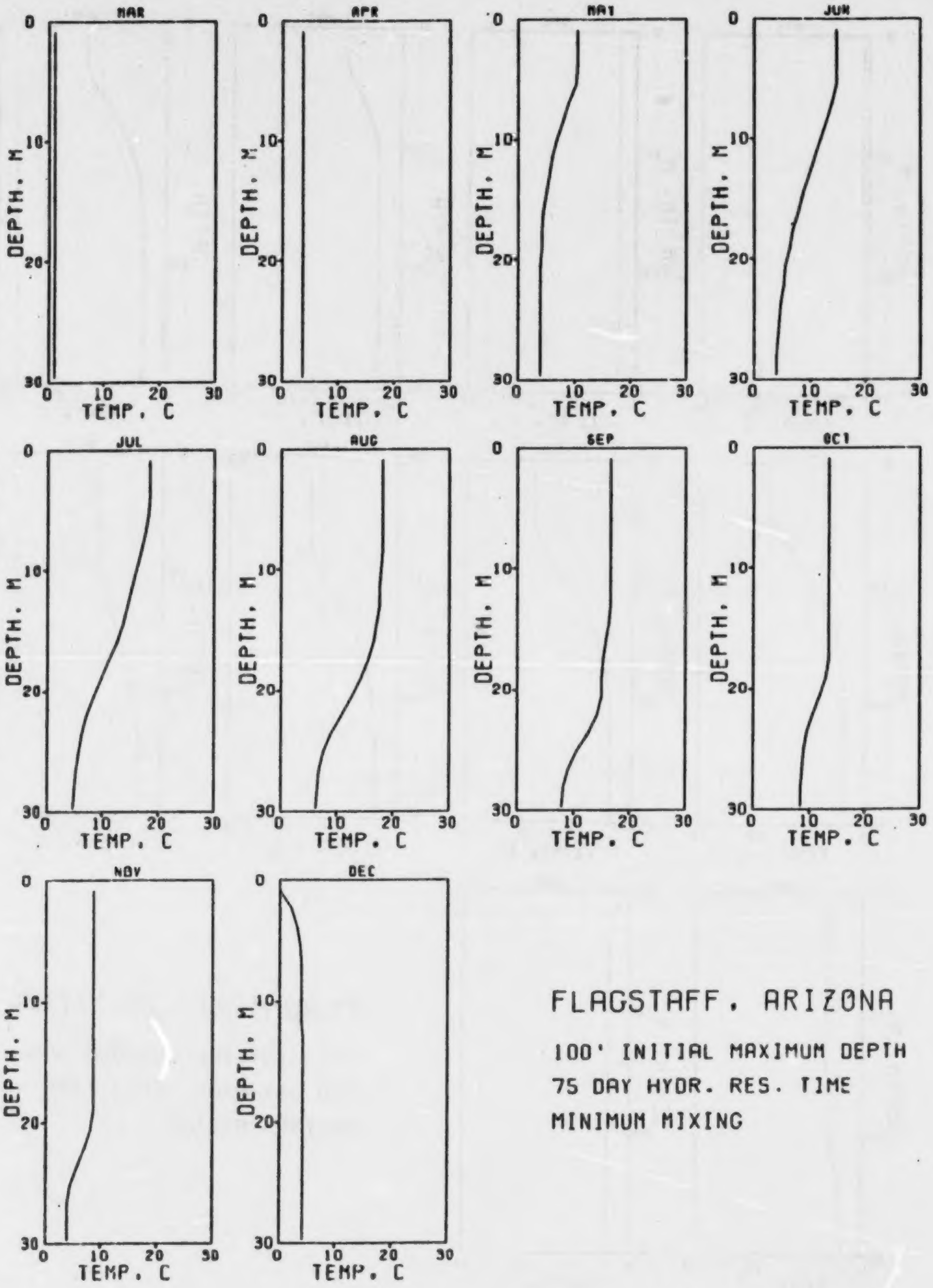
FLAGSTAFF, ARIZONA

100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



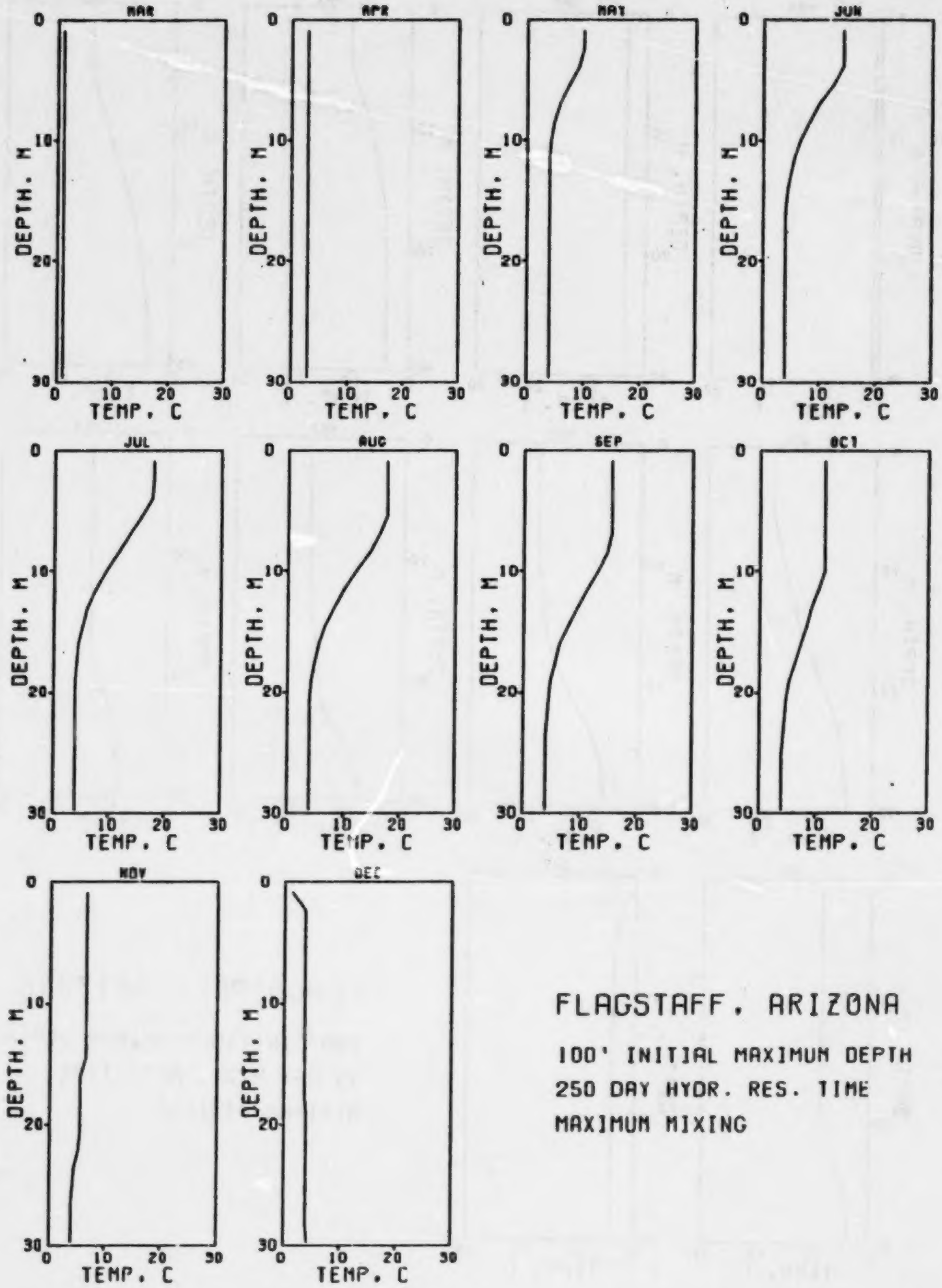
FLAGSTAFF, ARIZONA

100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



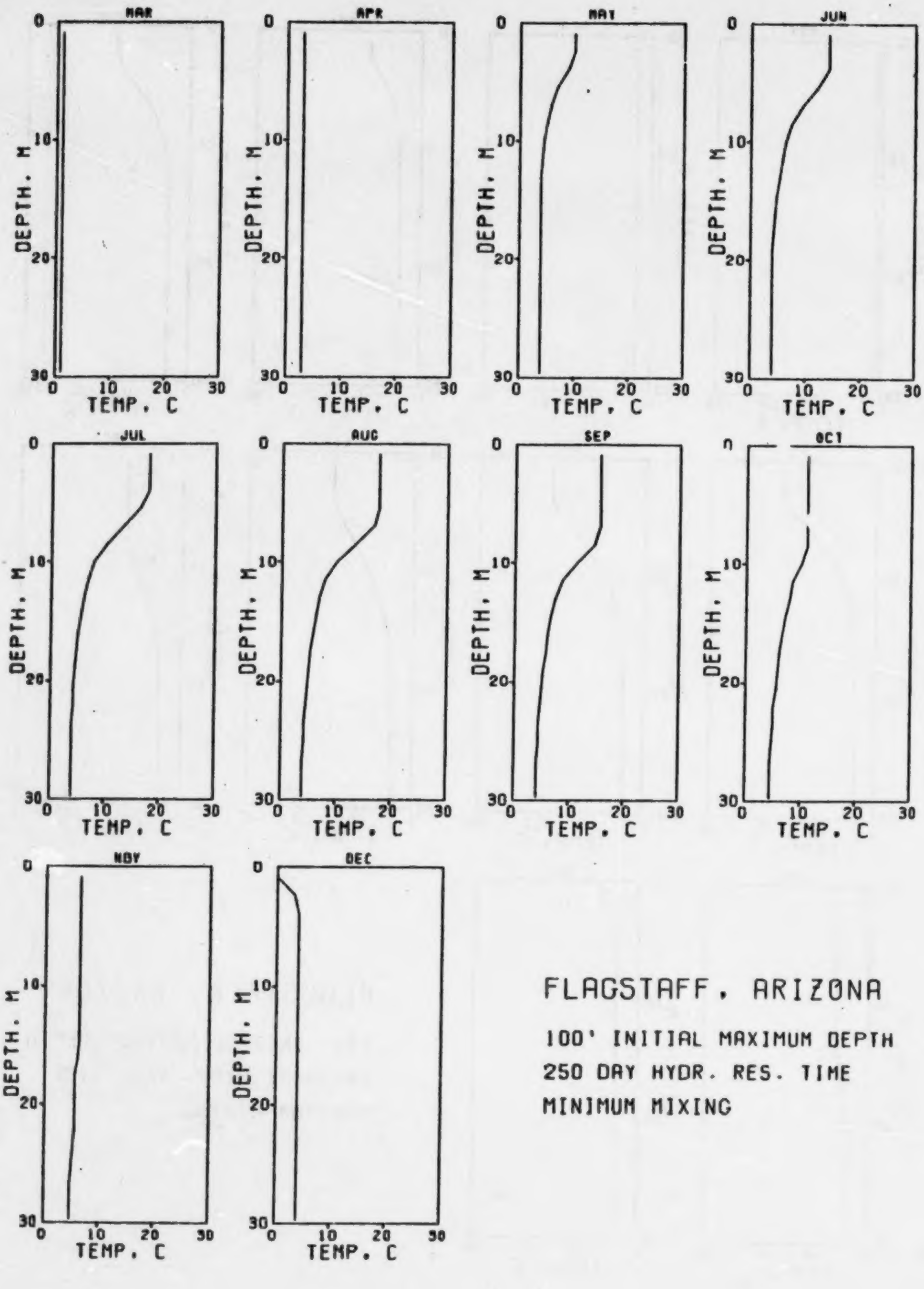
FLAGSTAFF, ARIZONA

100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



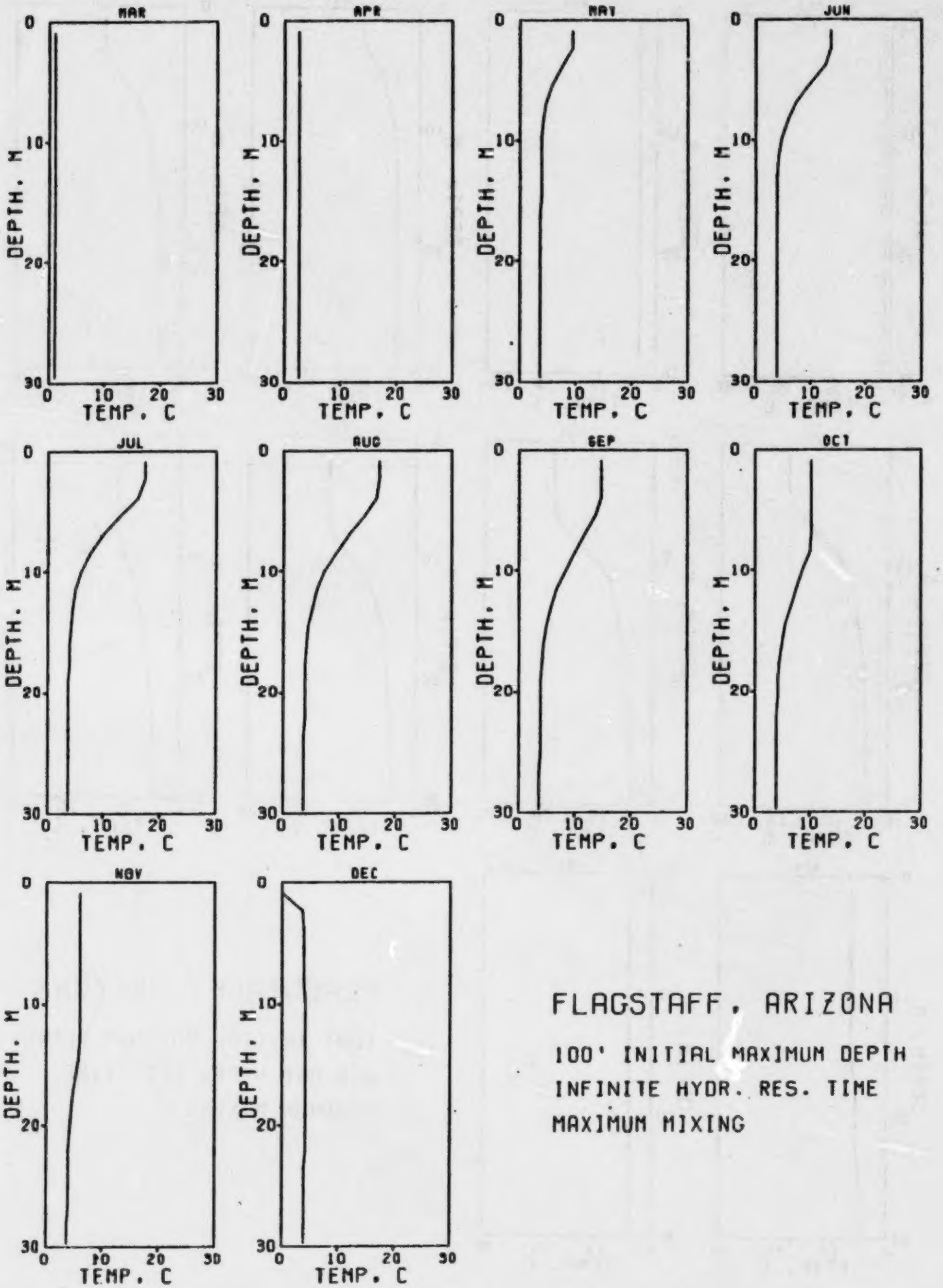
FLAGSTAFF, ARIZONA

100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



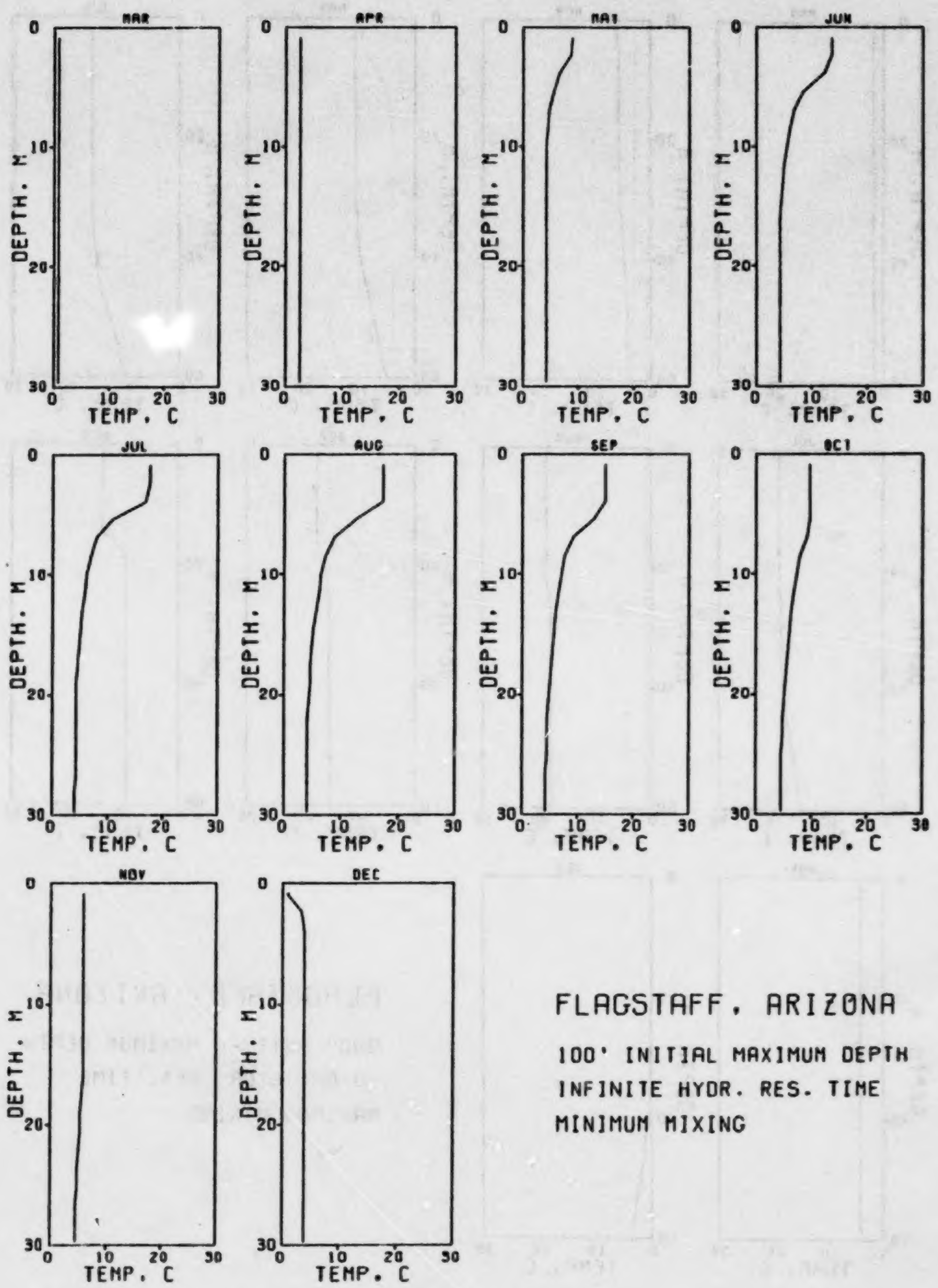
FLAGSTAFF, ARIZONA

100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



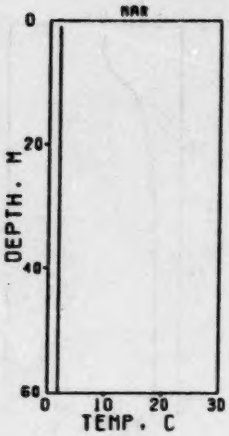
FLAGSTAFF, ARIZONA

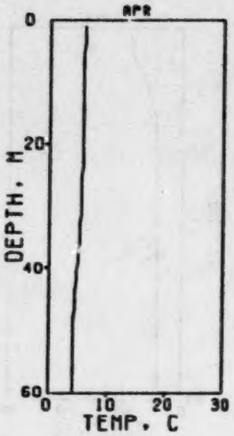
100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING

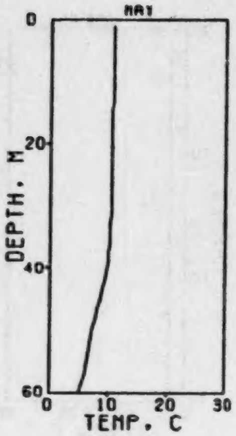


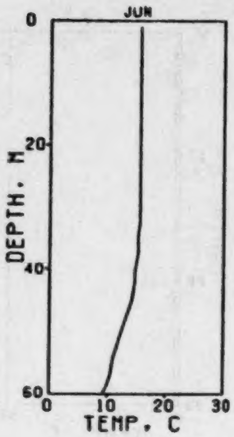
FLAGSTAFF, ARIZONA

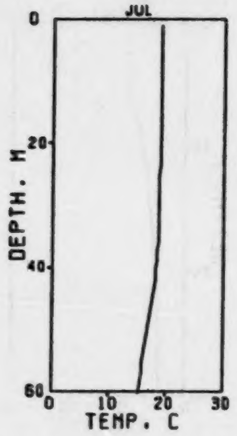
100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING

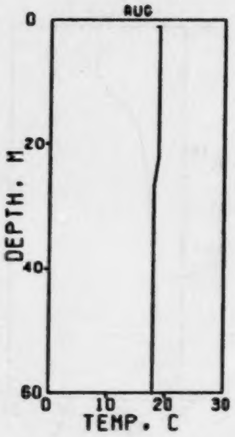


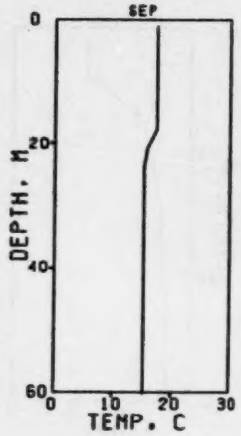


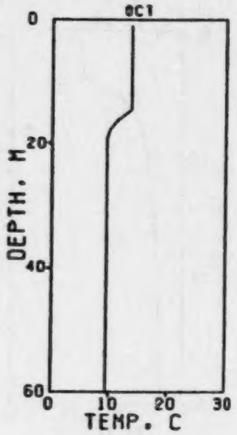


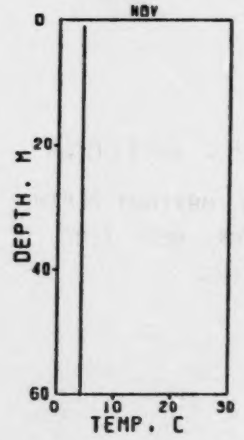


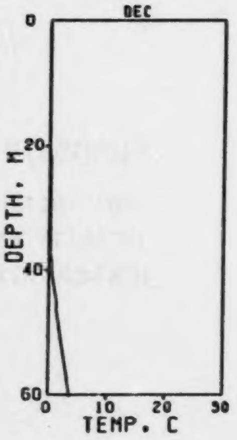






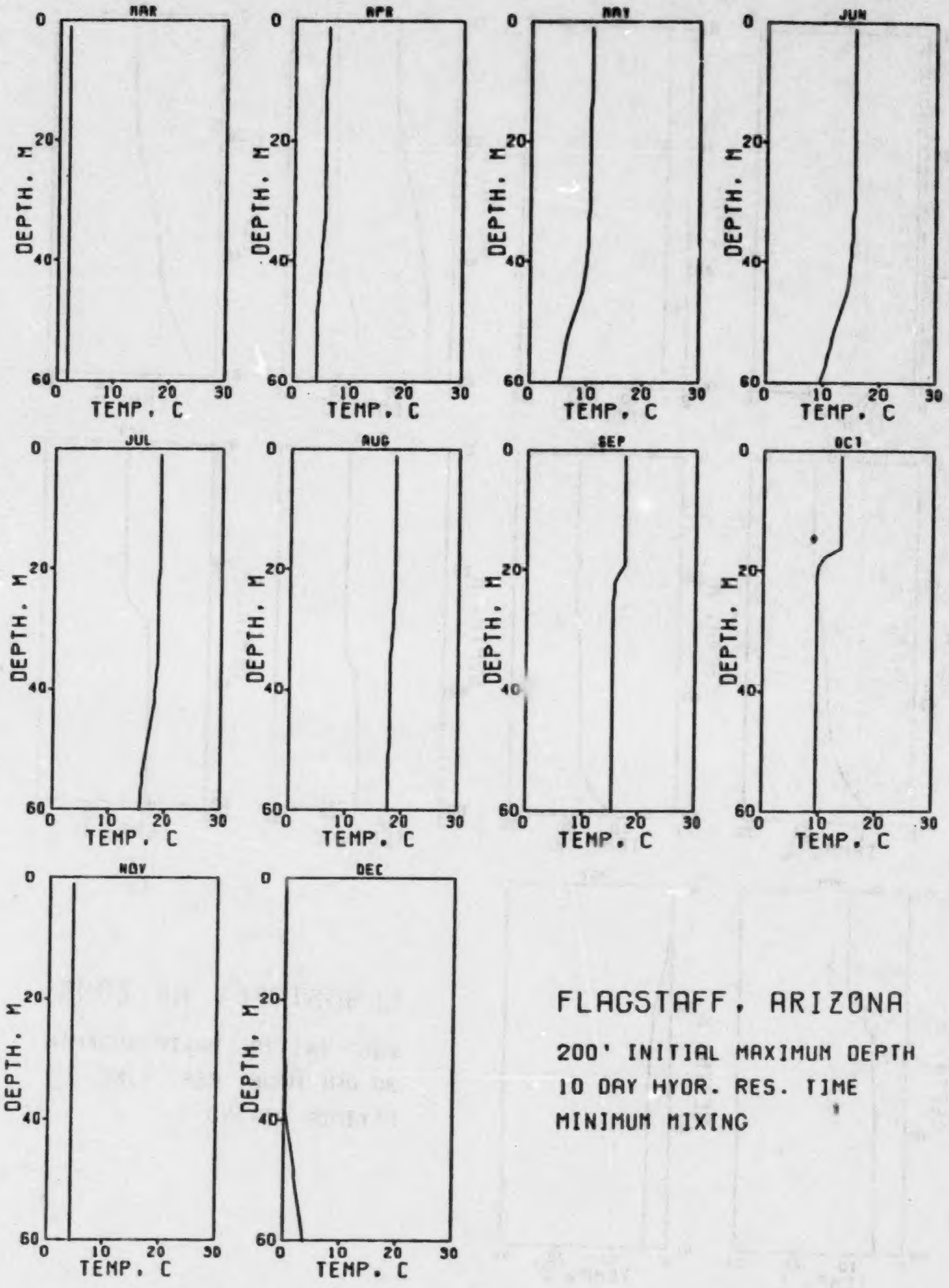






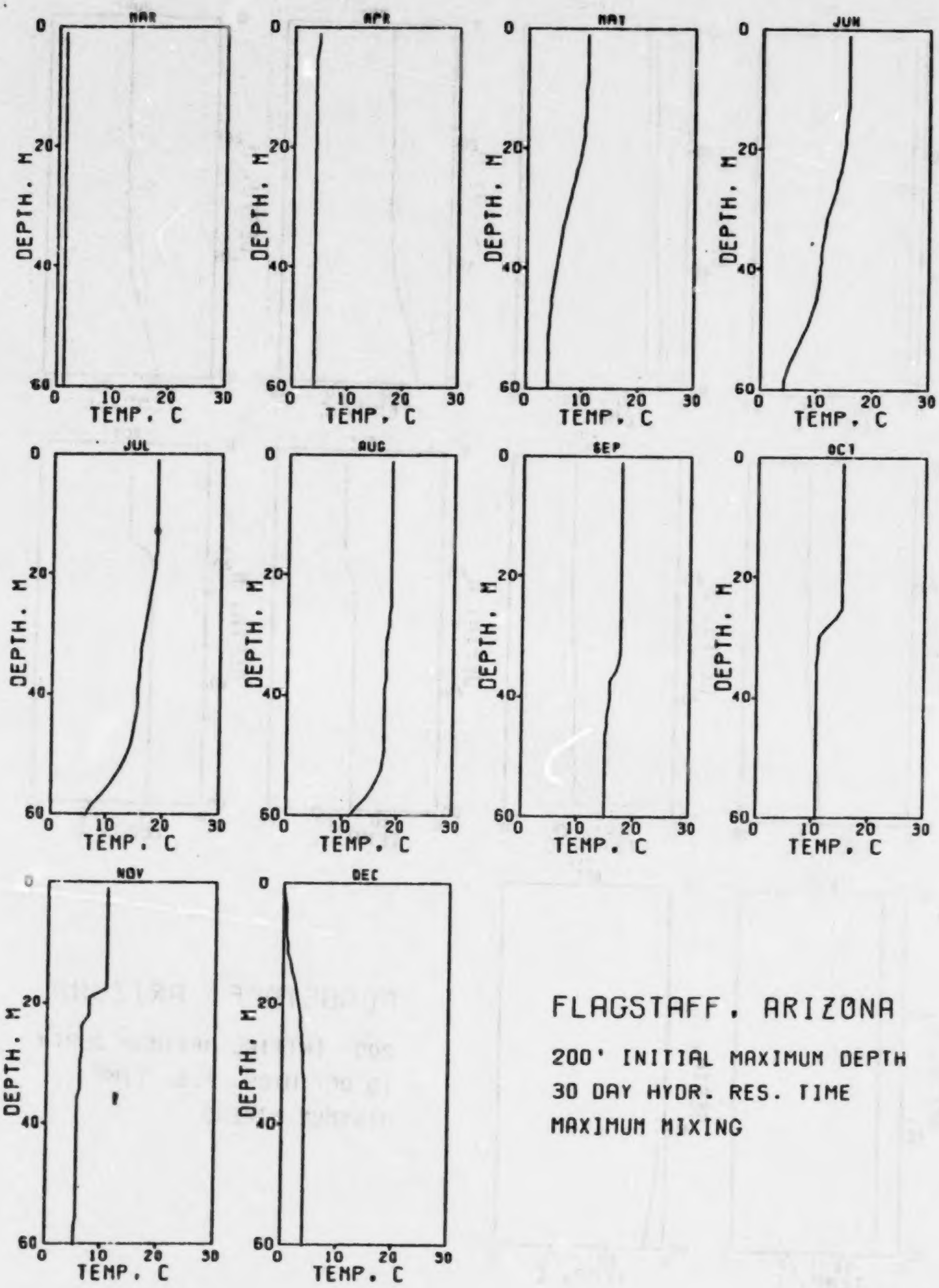
FLAGSTAFF, ARIZONA

200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



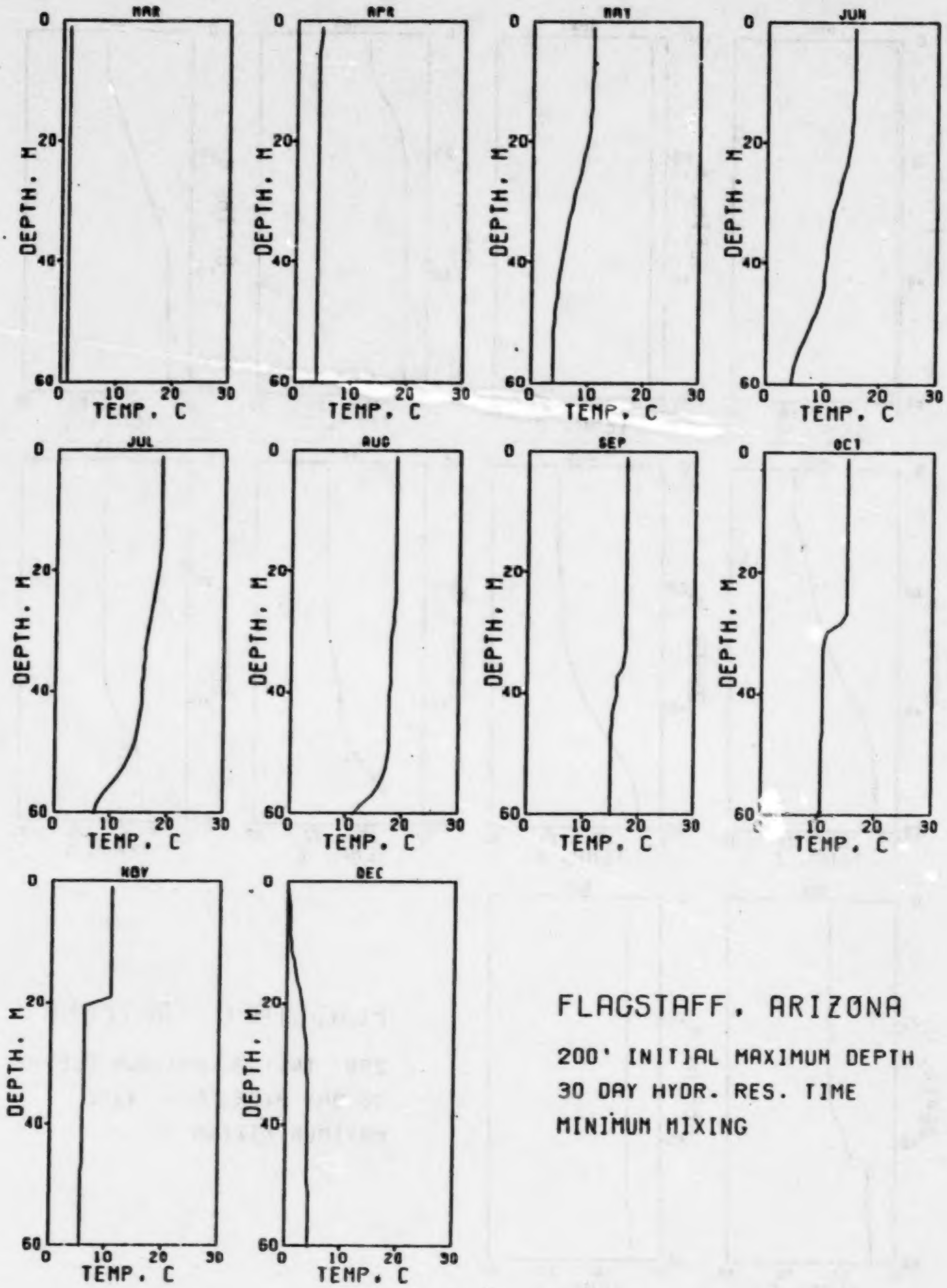
FLAGSTAFF, ARIZONA

200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



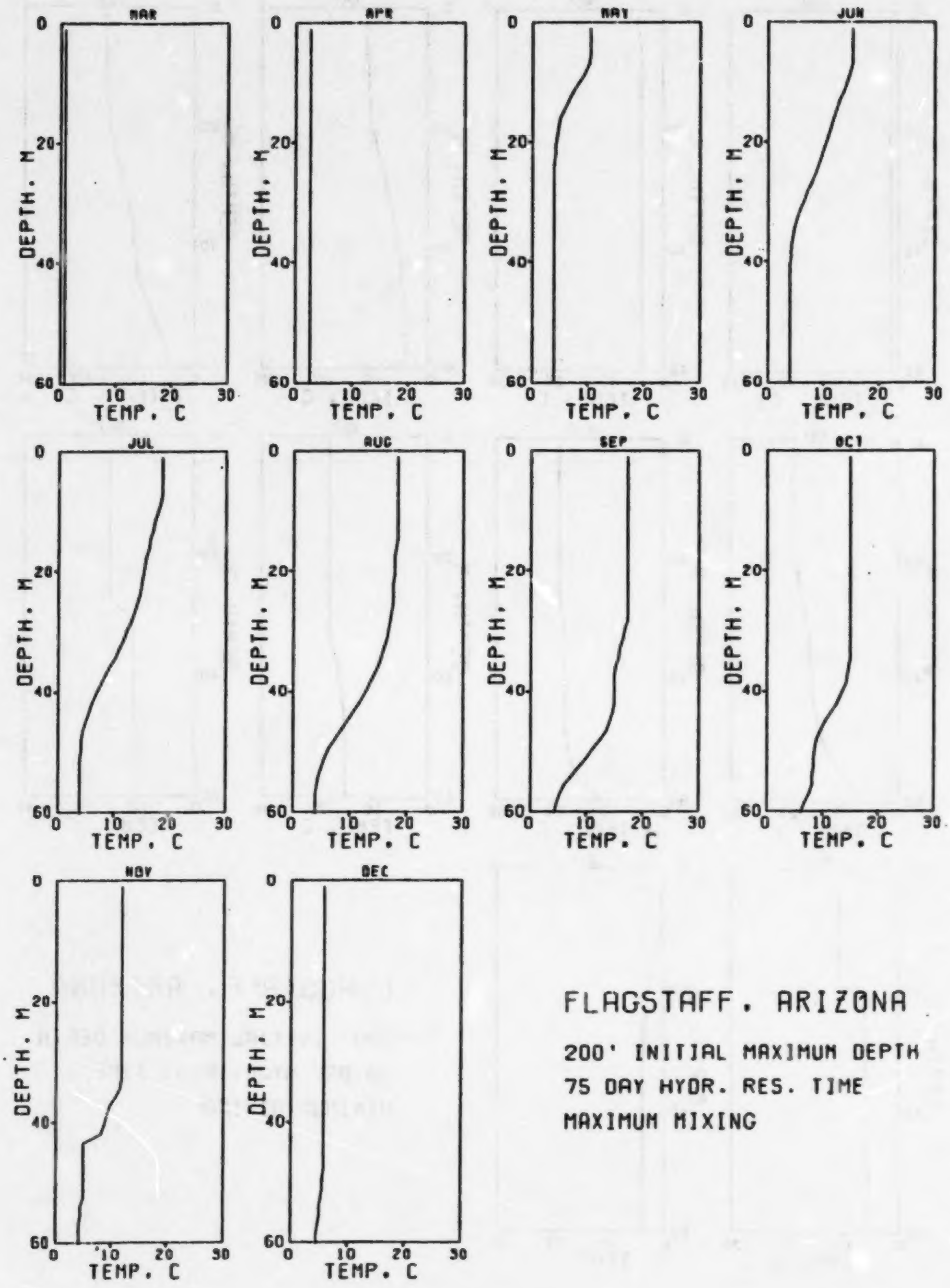
FLAGSTAFF, ARIZONA

200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



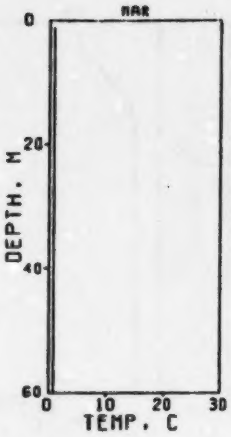
FLAGSTAFF, ARIZONA

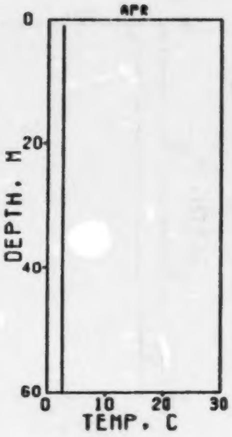
200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING

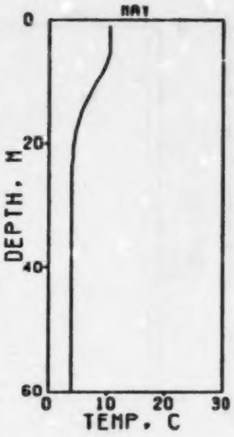


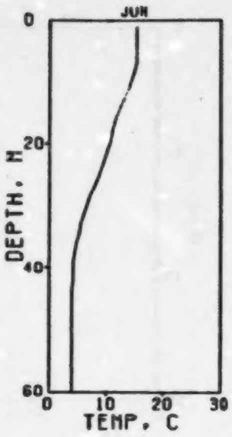
FLAGSTAFF, ARIZONA

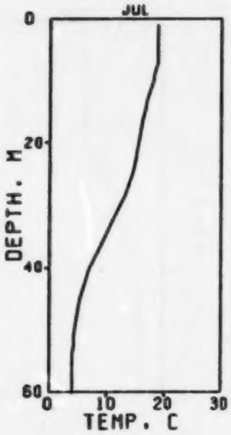
200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING

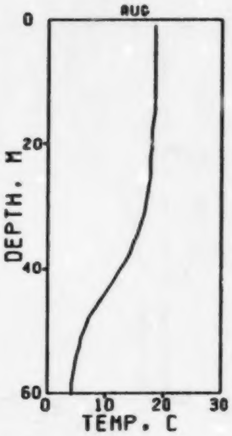


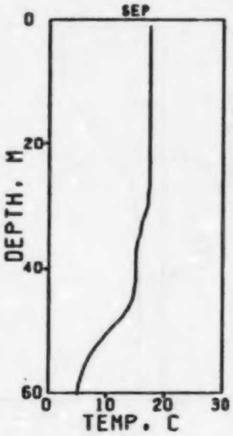


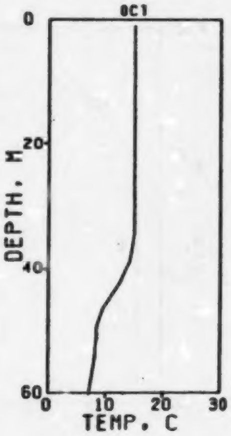


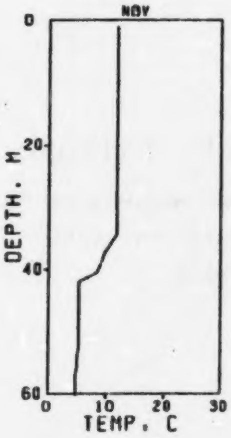


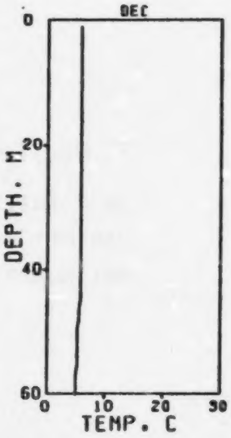






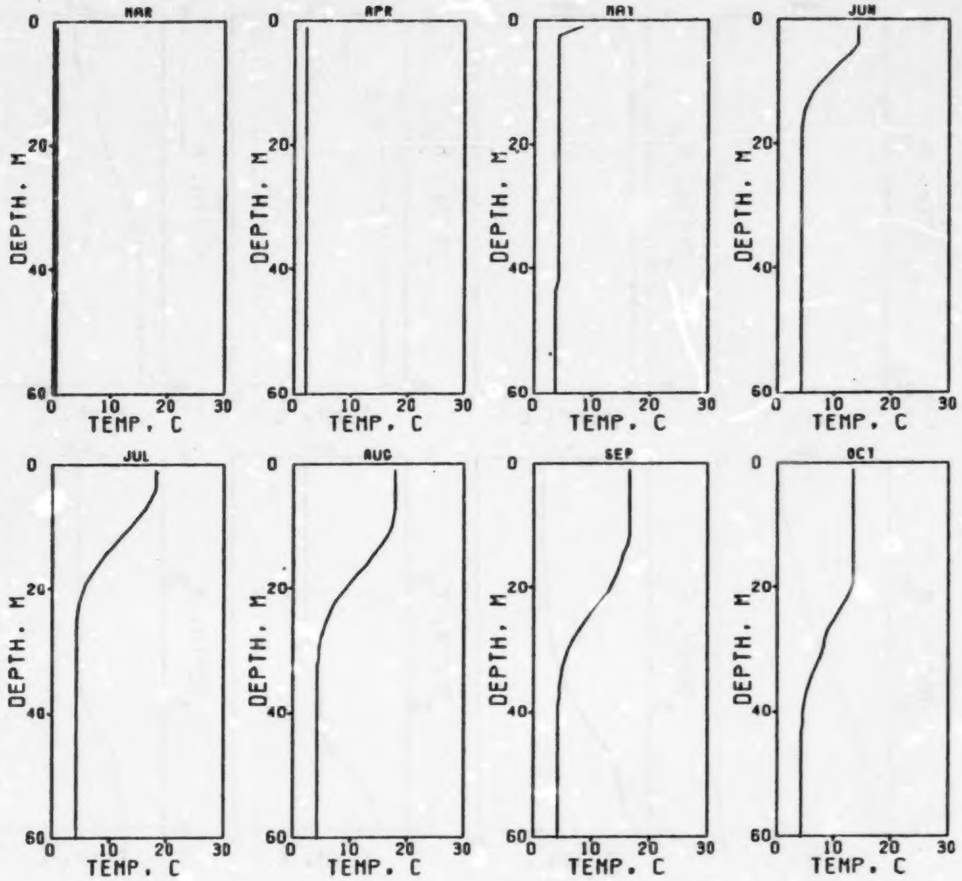


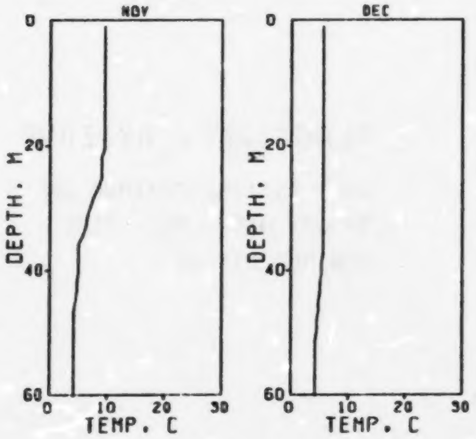




FLAGSTAFF, ARIZONA

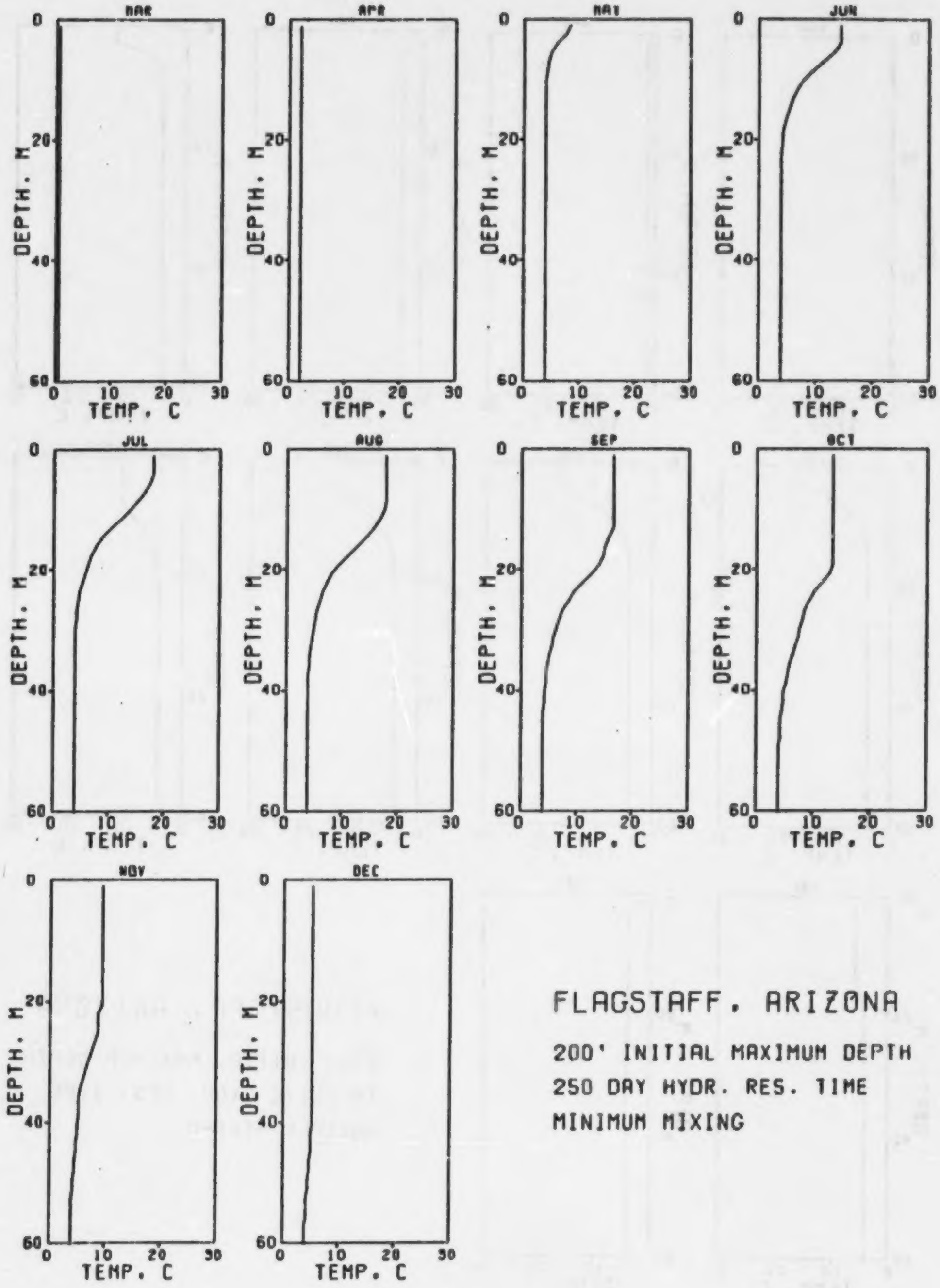
200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING





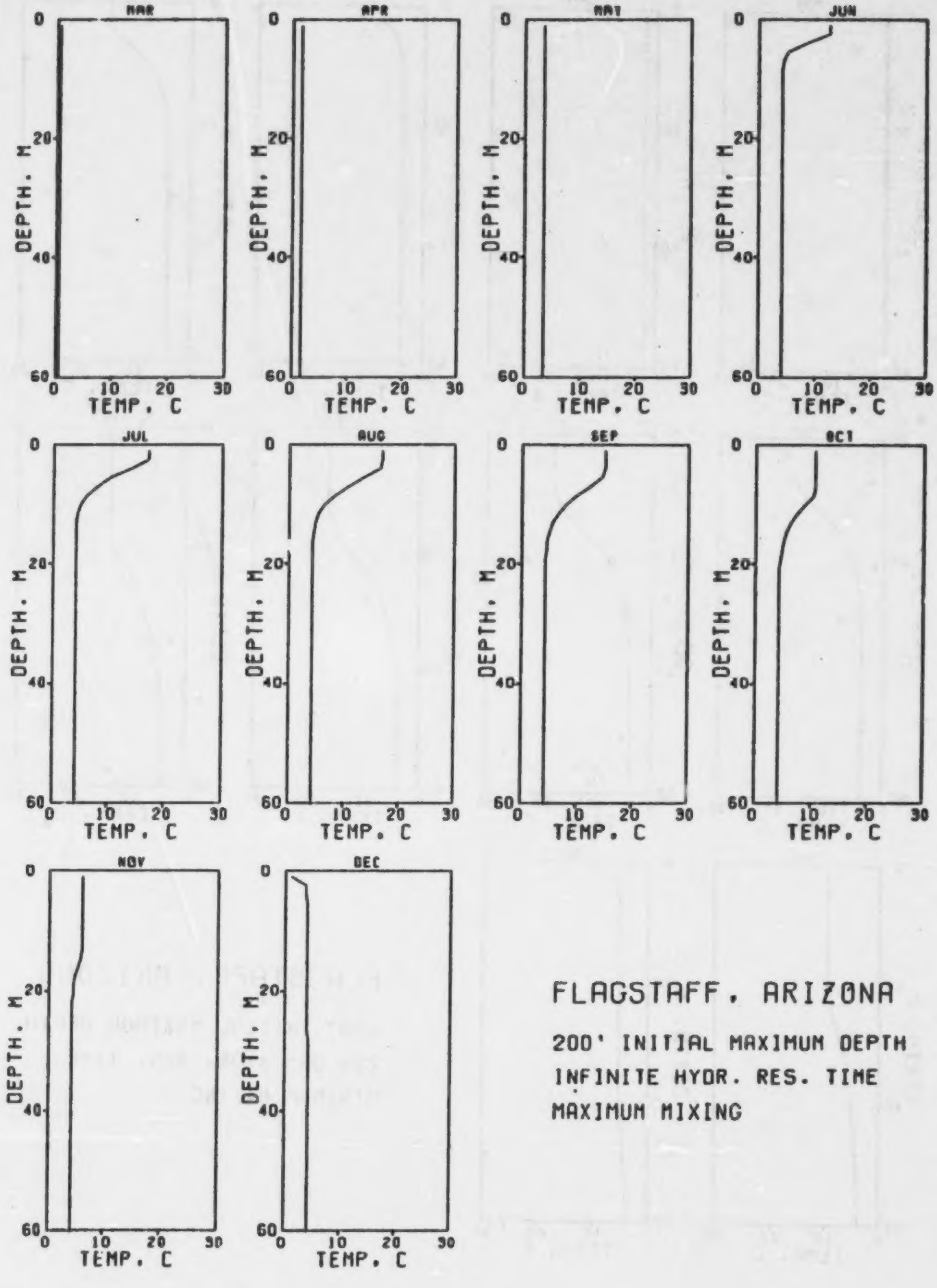
FLAGSTAFF, ARIZONA

200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



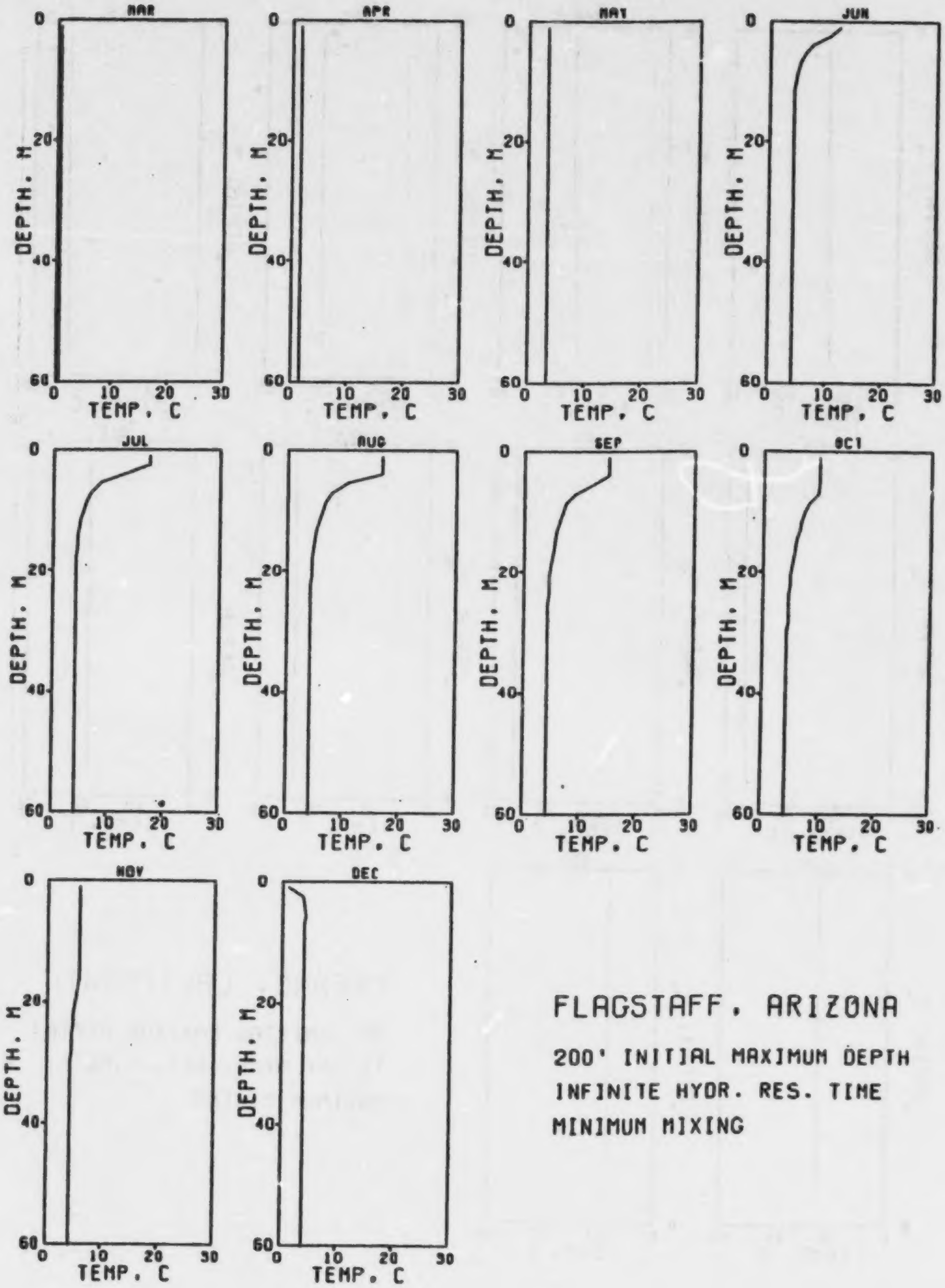
FLAGSTAFF, ARIZONA

200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



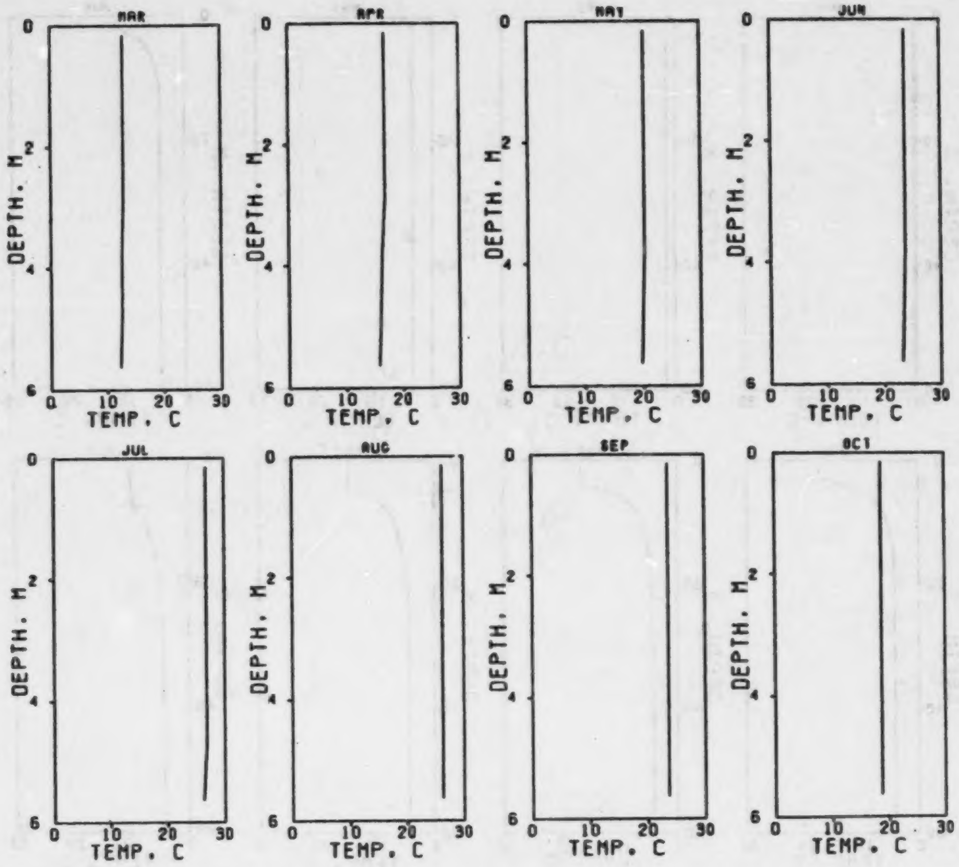
FLAGSTAFF, ARIZONA

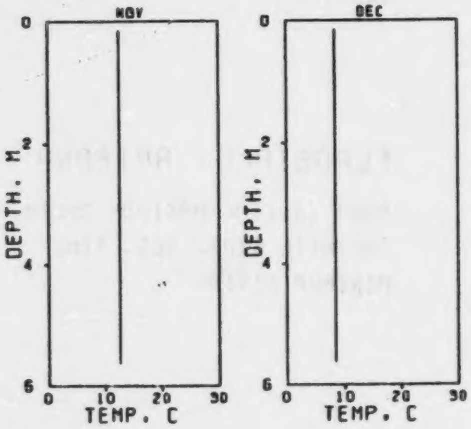
200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



FLAGSTAFF, ARIZONA

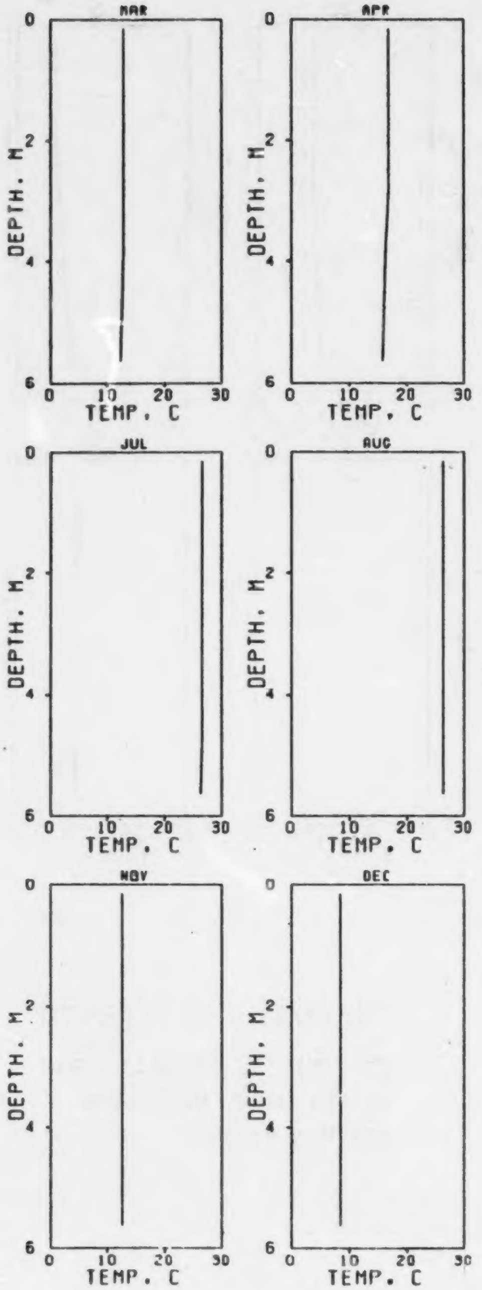
200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING





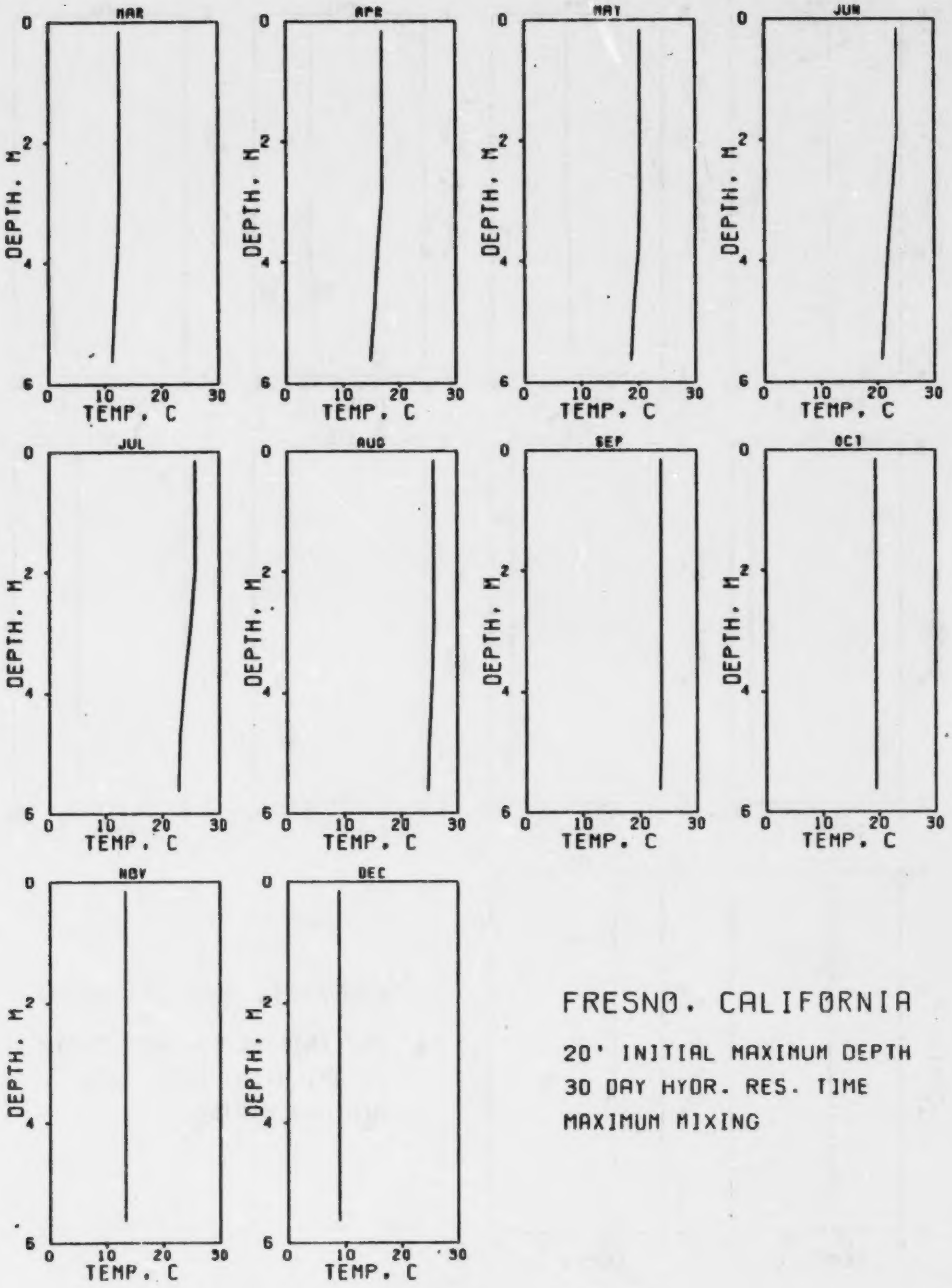
FRESNO, CALIFORNIA

20' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



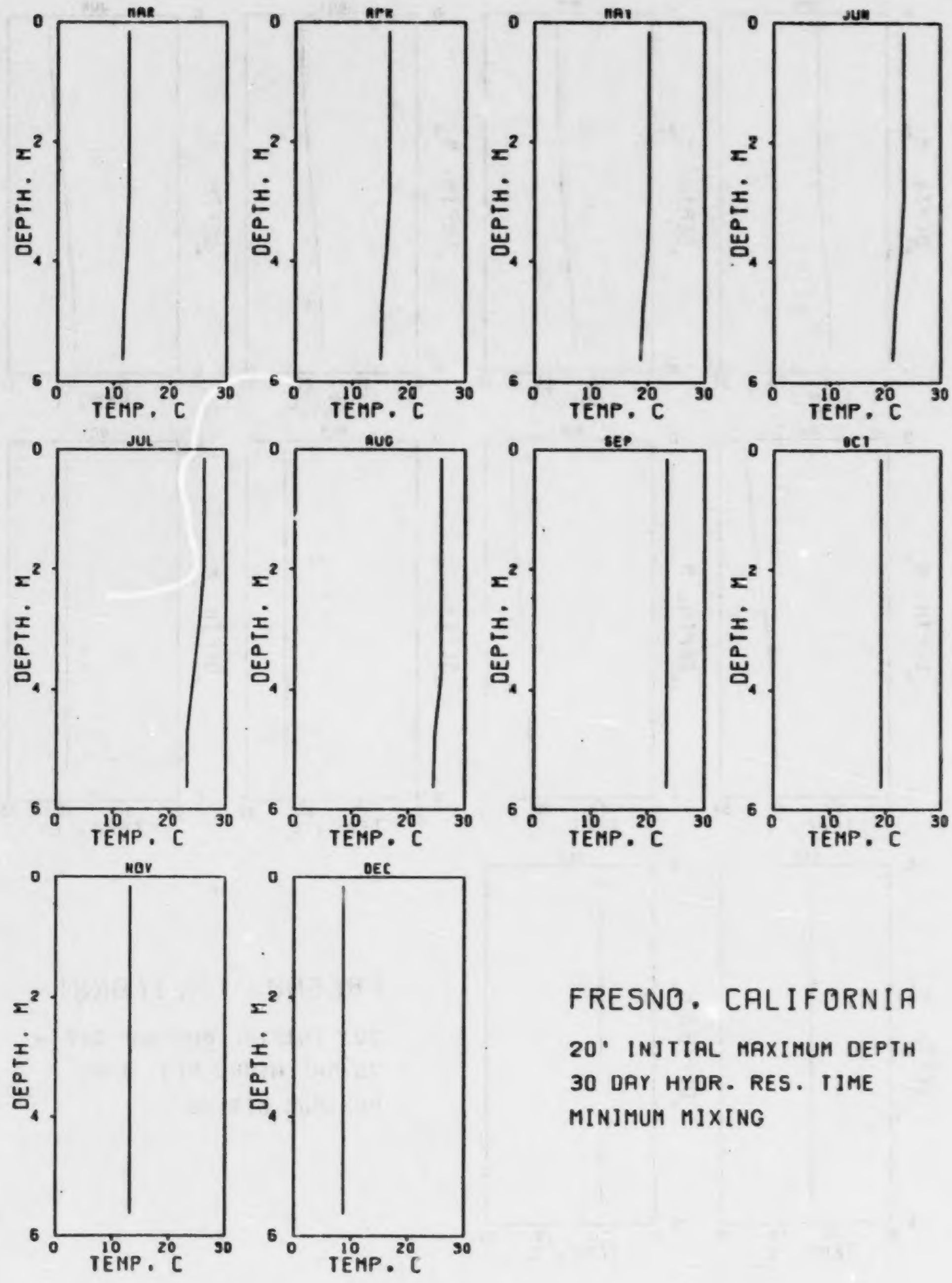
FRESNO, CALIFORNIA

20' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



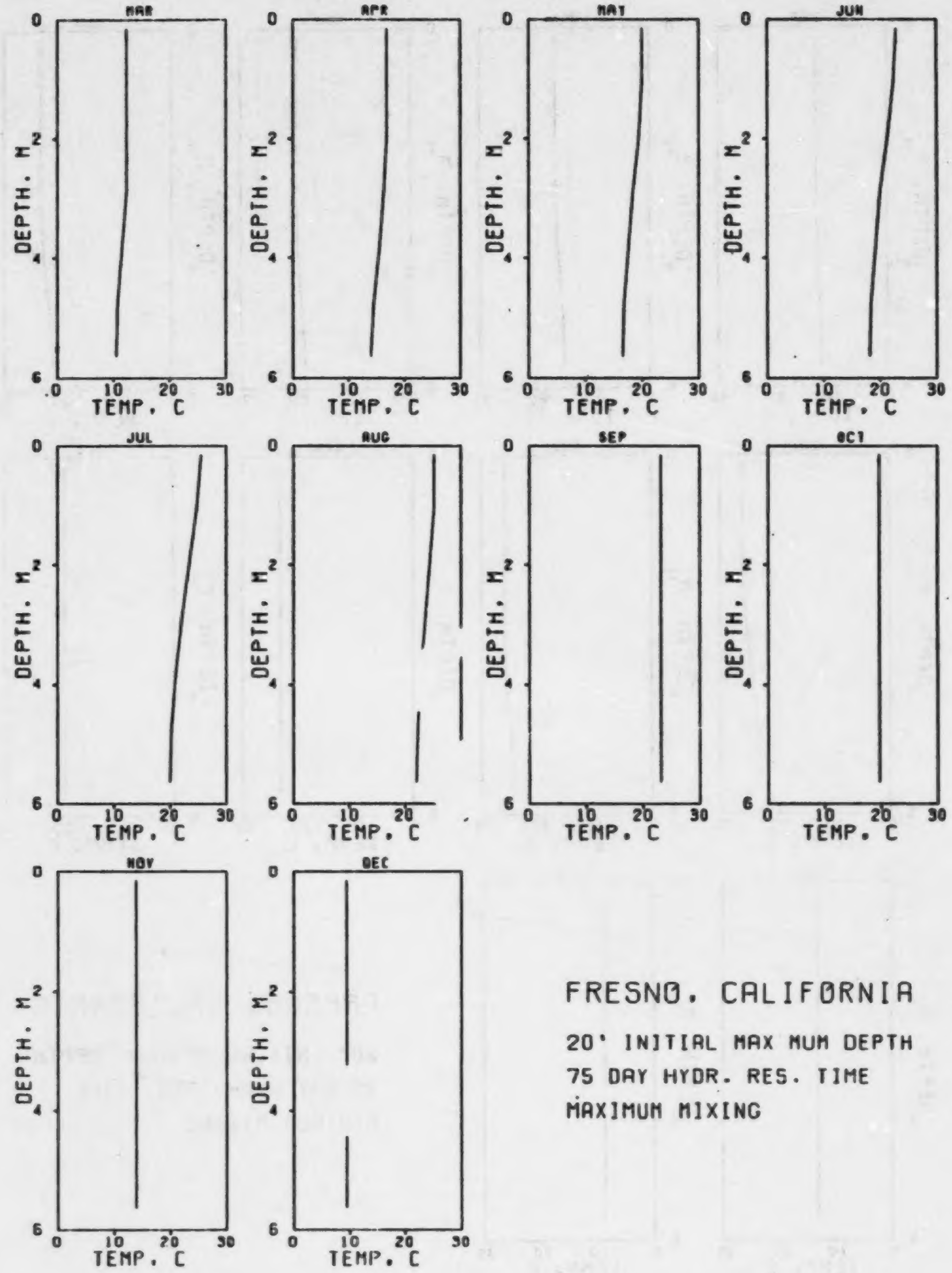
FRESNO, CALIFORNIA

20' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



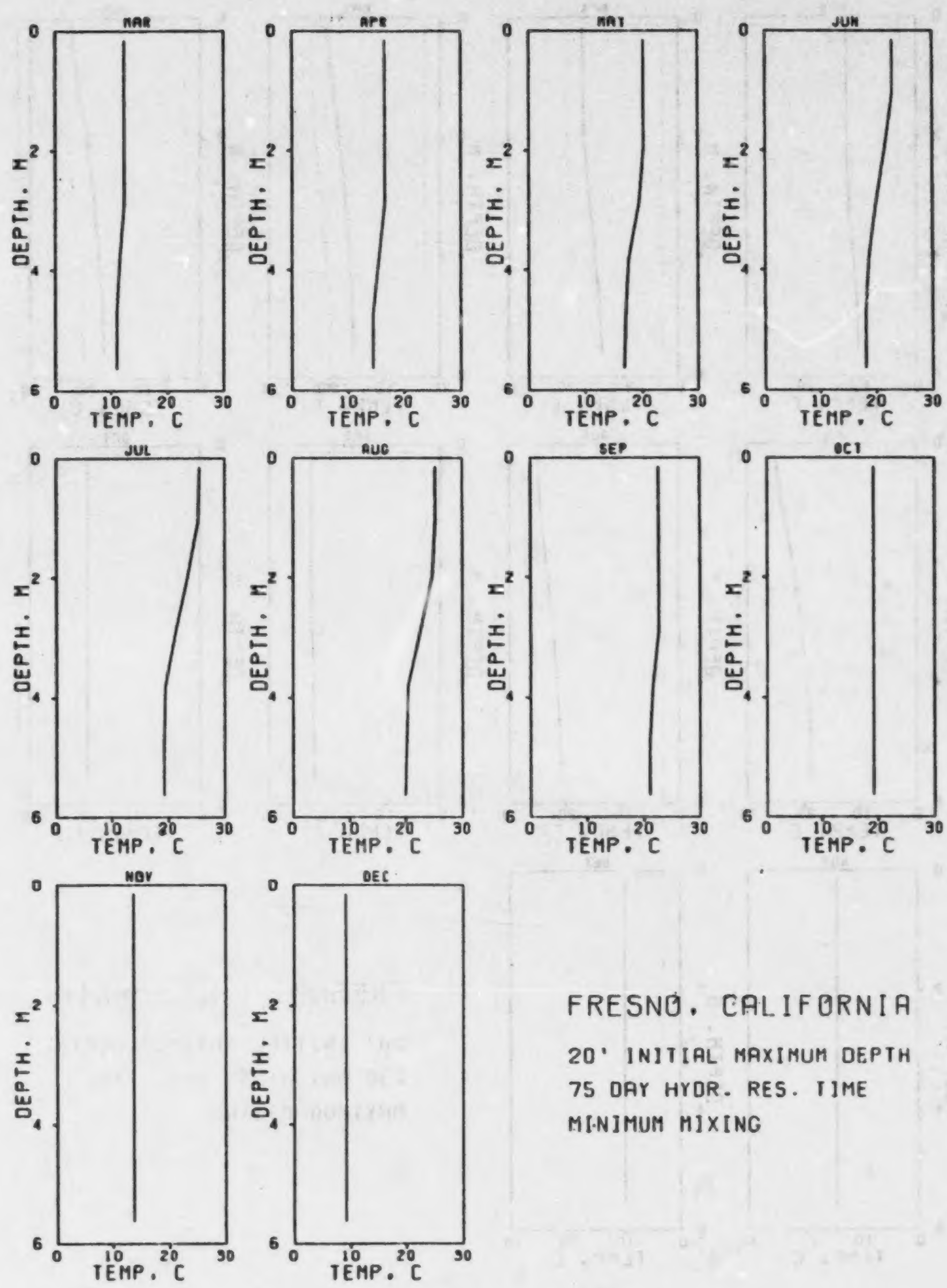
FRESNO, CALIFORNIA

20' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



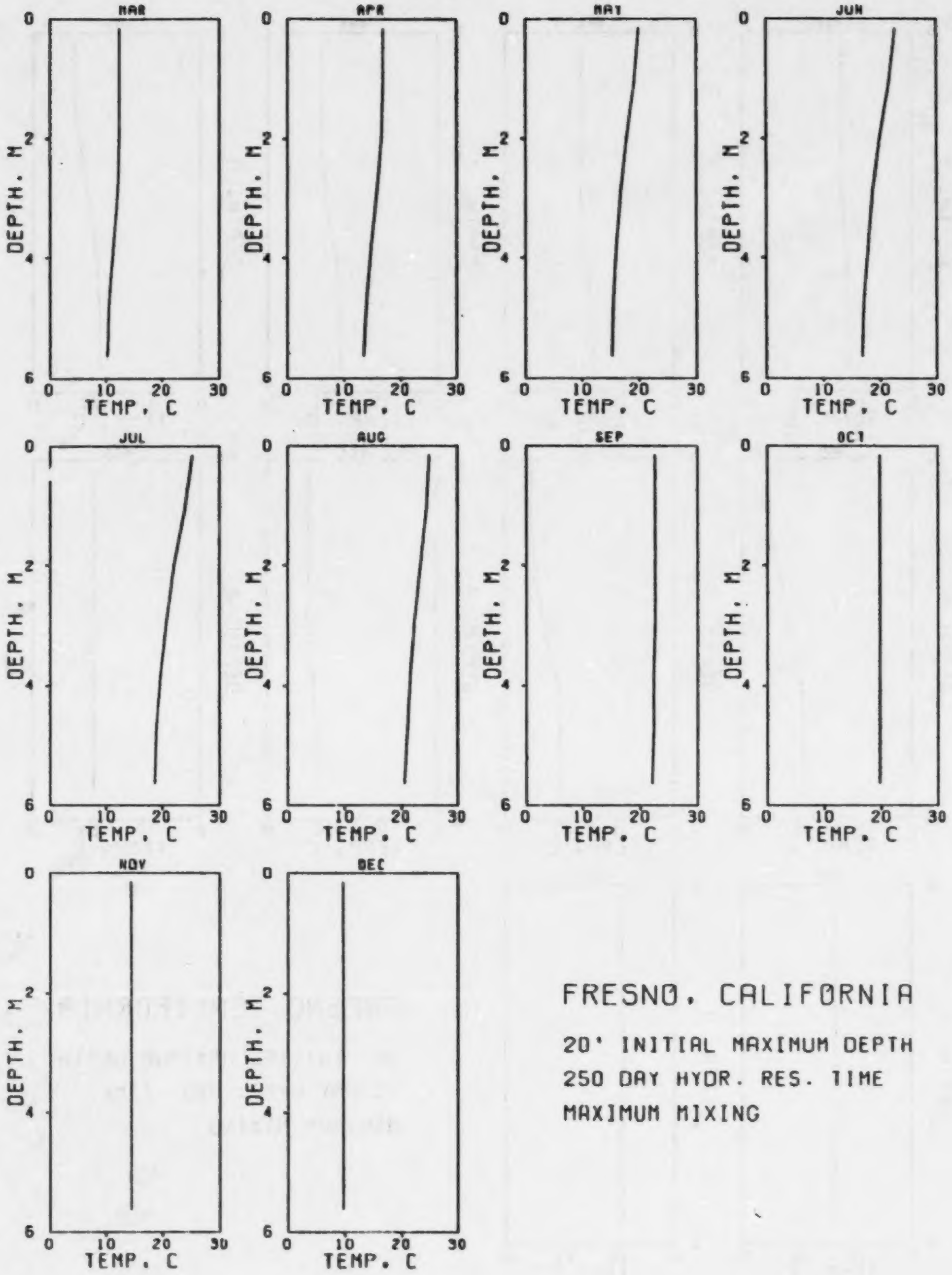
FRESNO, CALIFORNIA

20' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



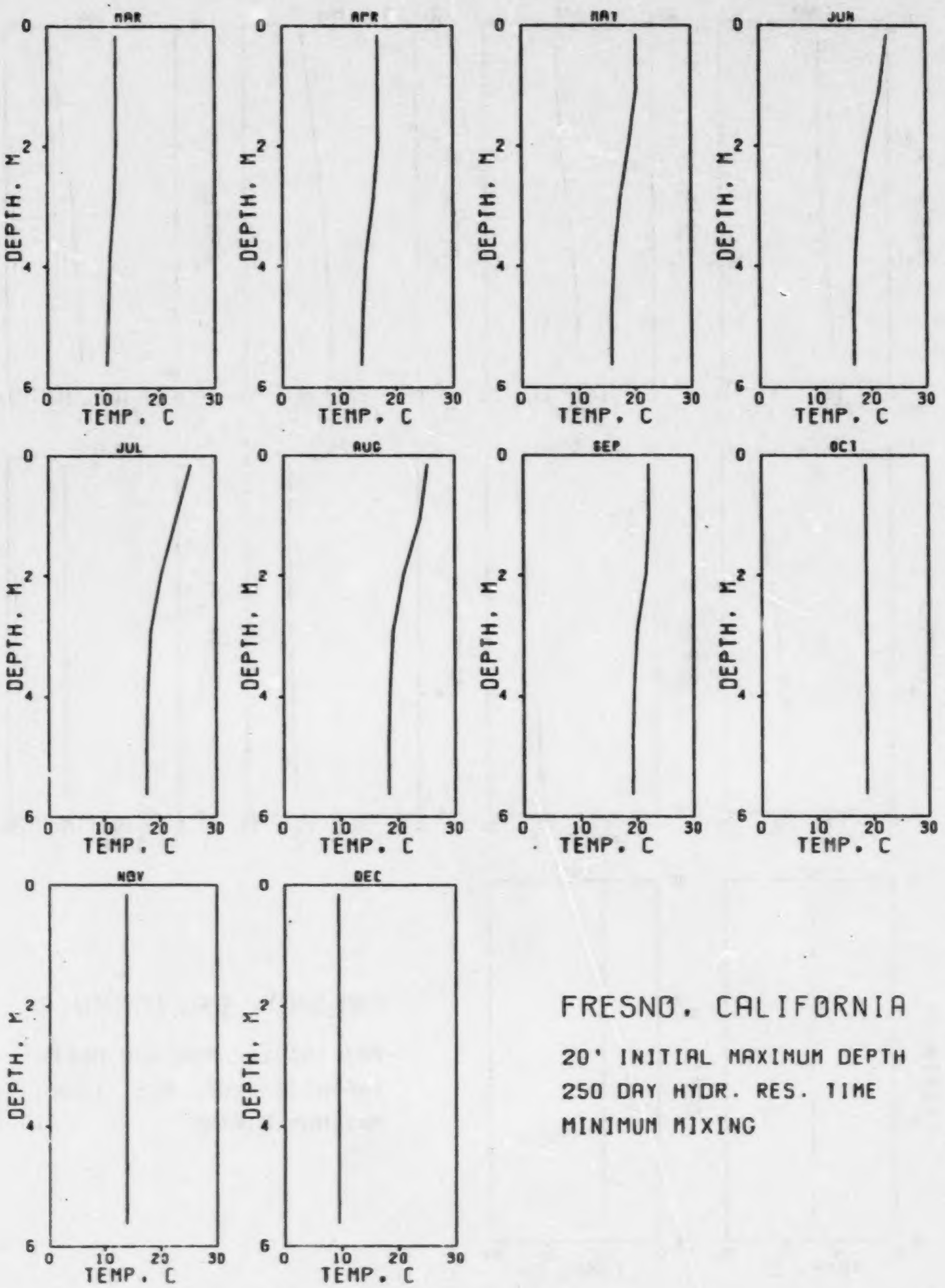
FRESNO, CALIFORNIA

20' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



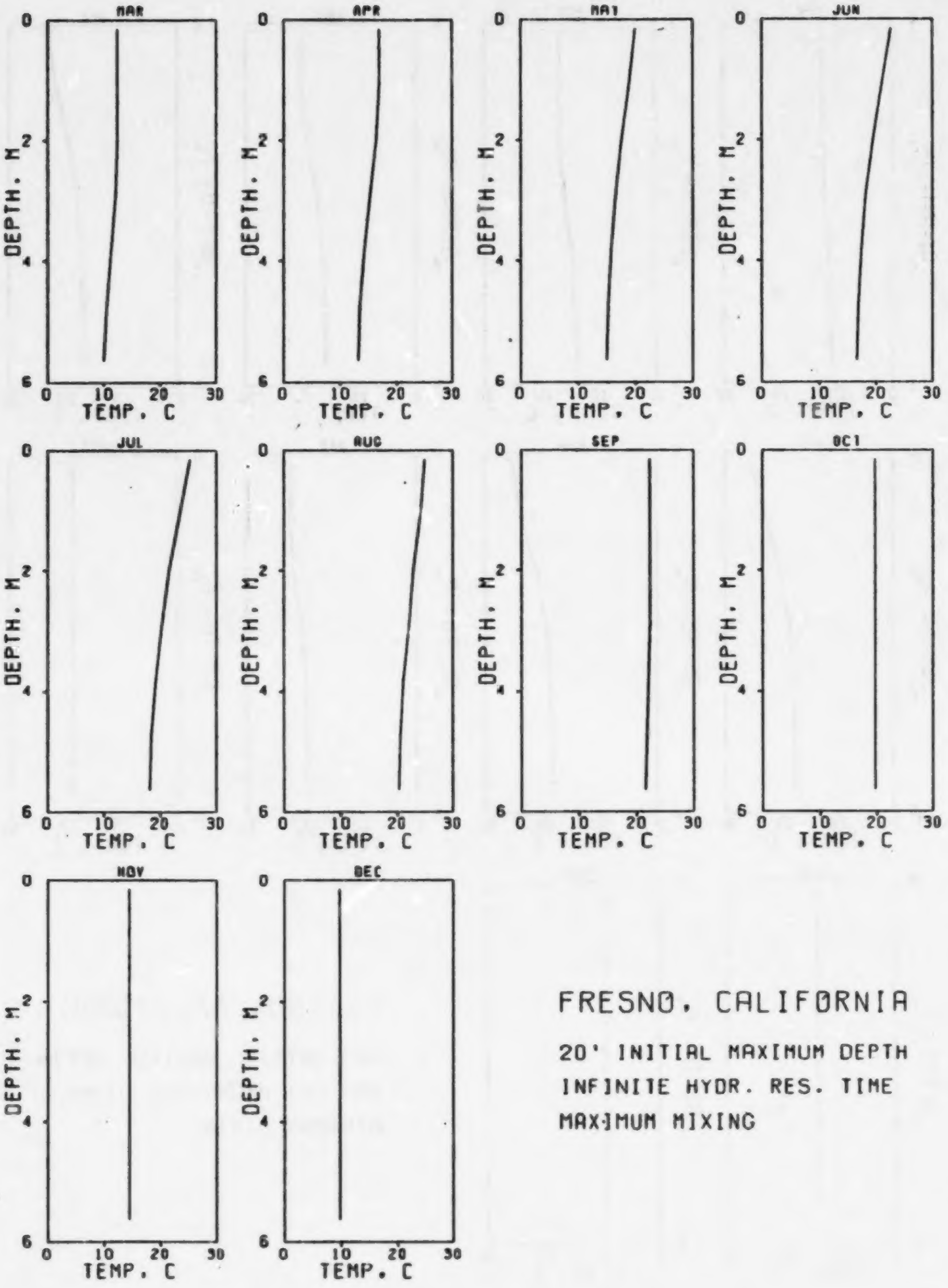
FRESNO, CALIFORNIA

20' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



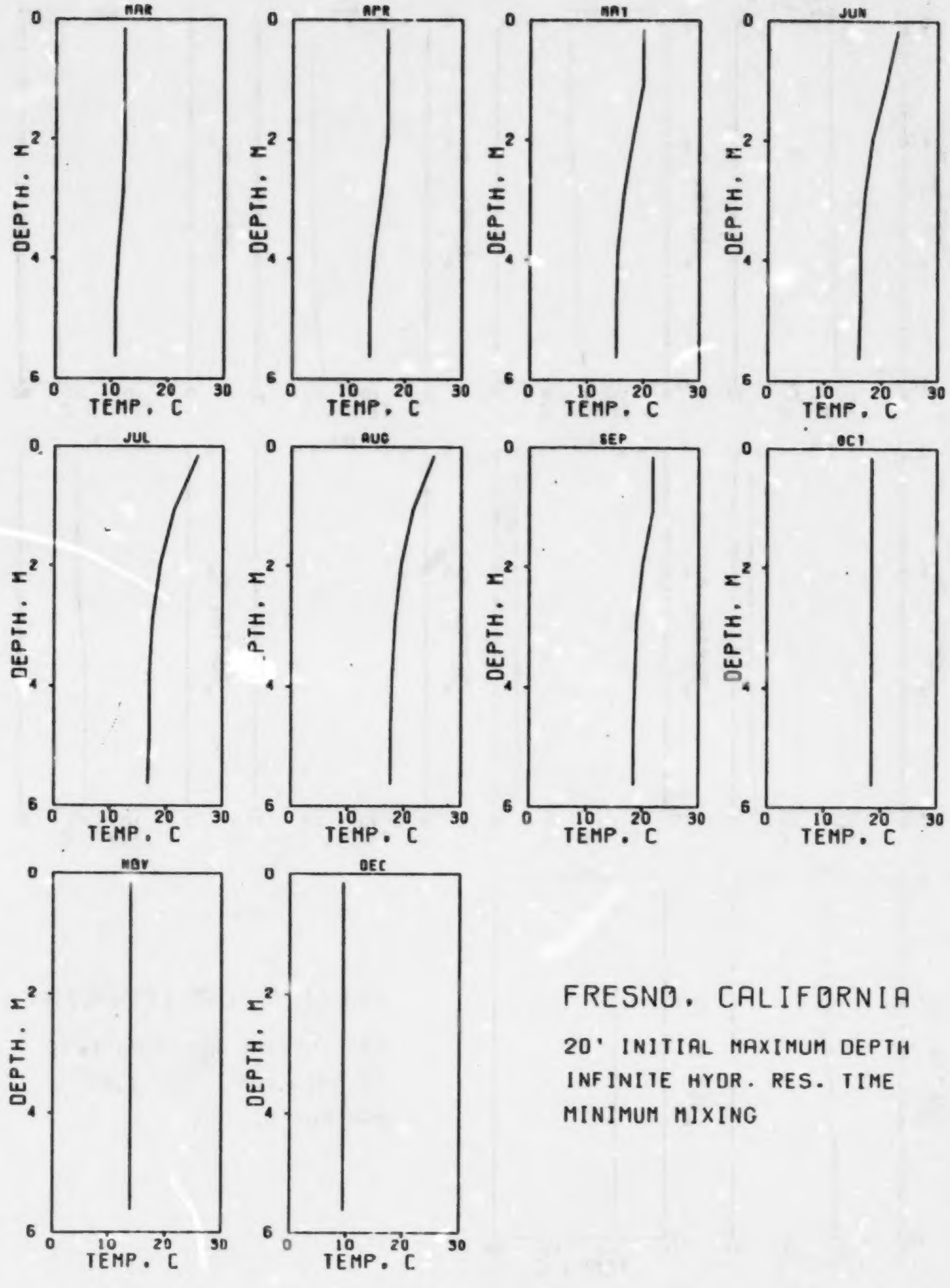
FRESNO, CALIFORNIA

20' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



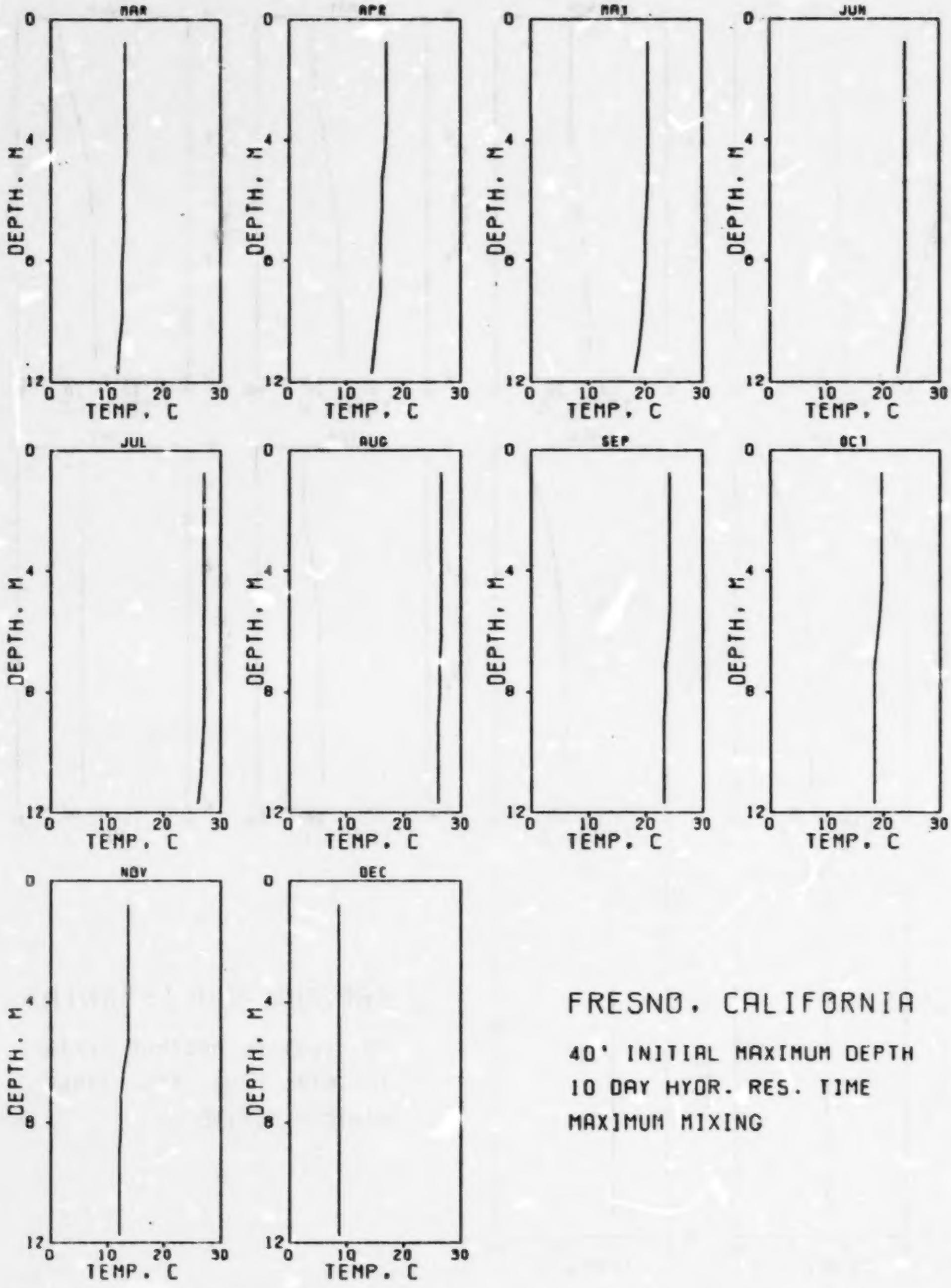
FRESNO, CALIFORNIA

20' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



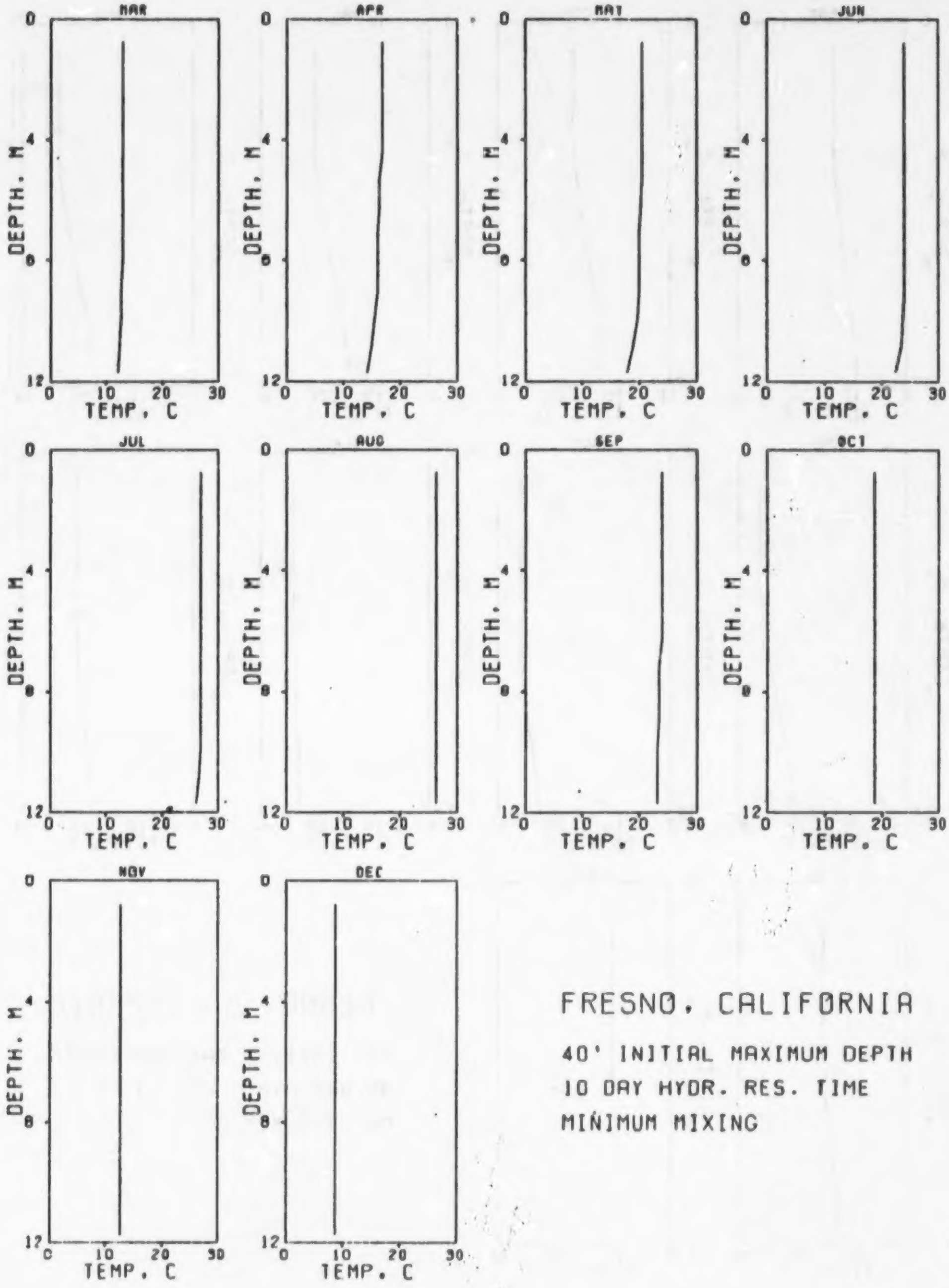
FRESNO, CALIFORNIA

20' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



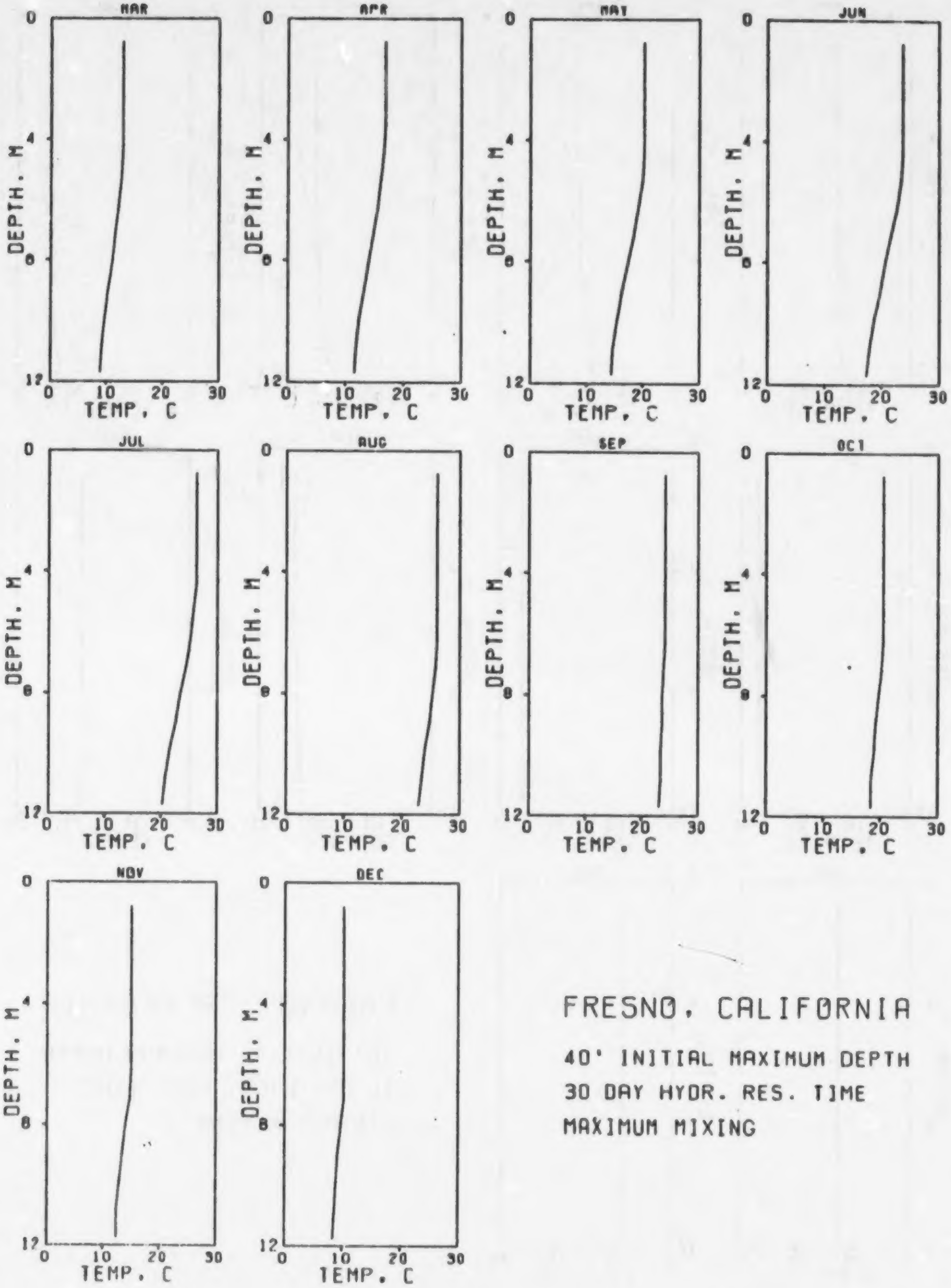
FRESNO, CALIFORNIA

40' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



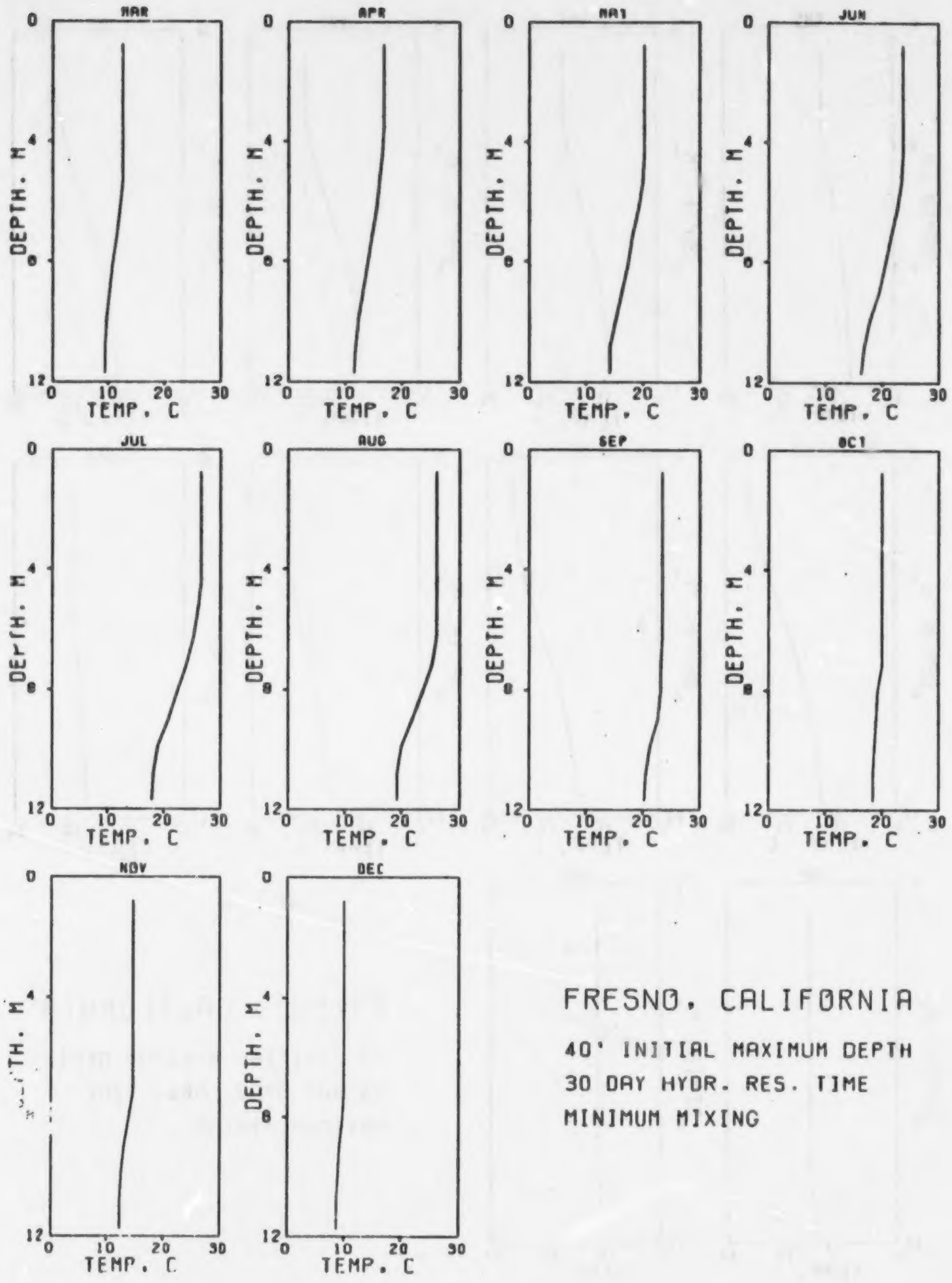
FRESNO, CALIFORNIA

40' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



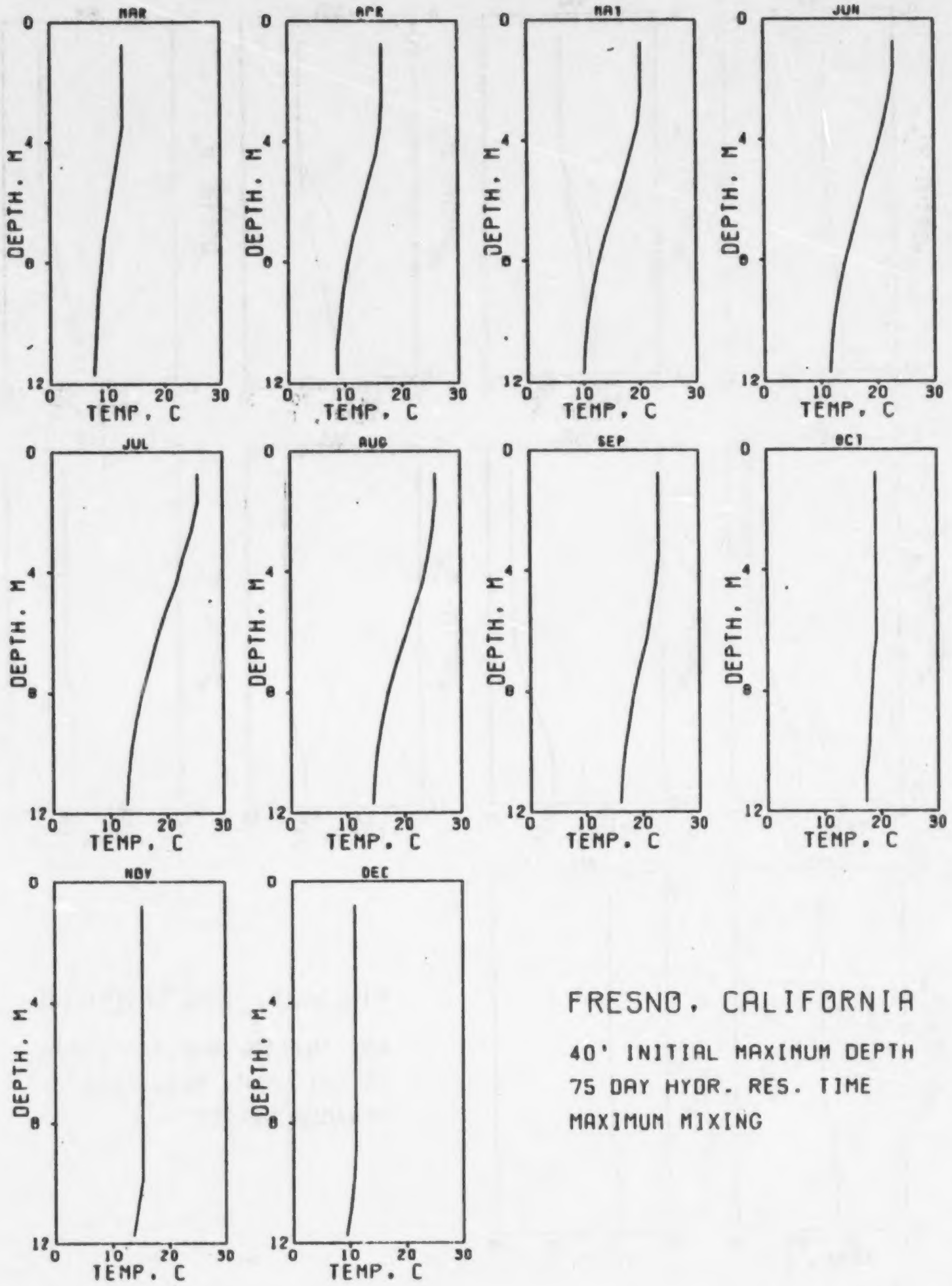
FRESNO, CALIFORNIA

40' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



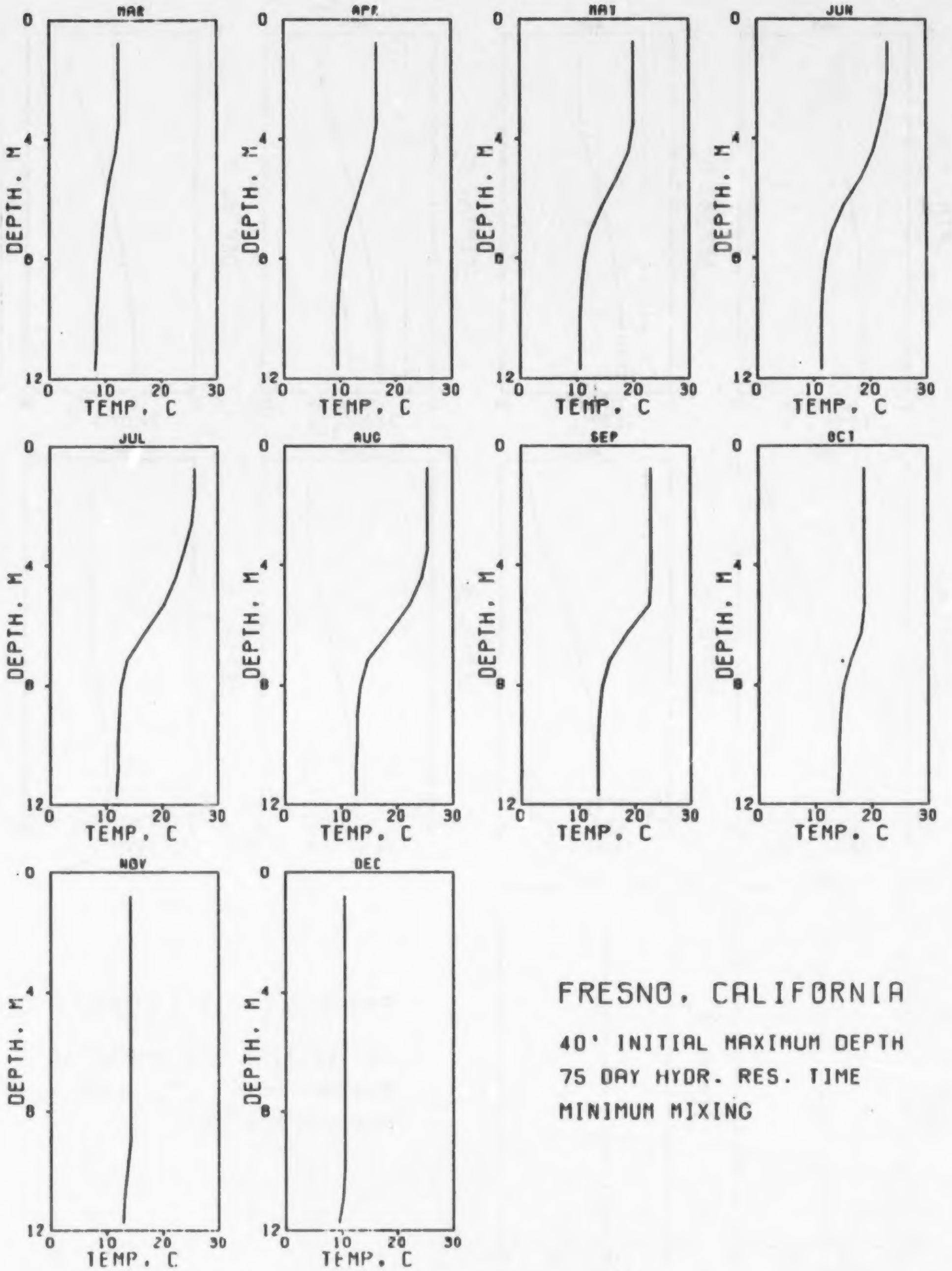
FRESNO, CALIFORNIA

40' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



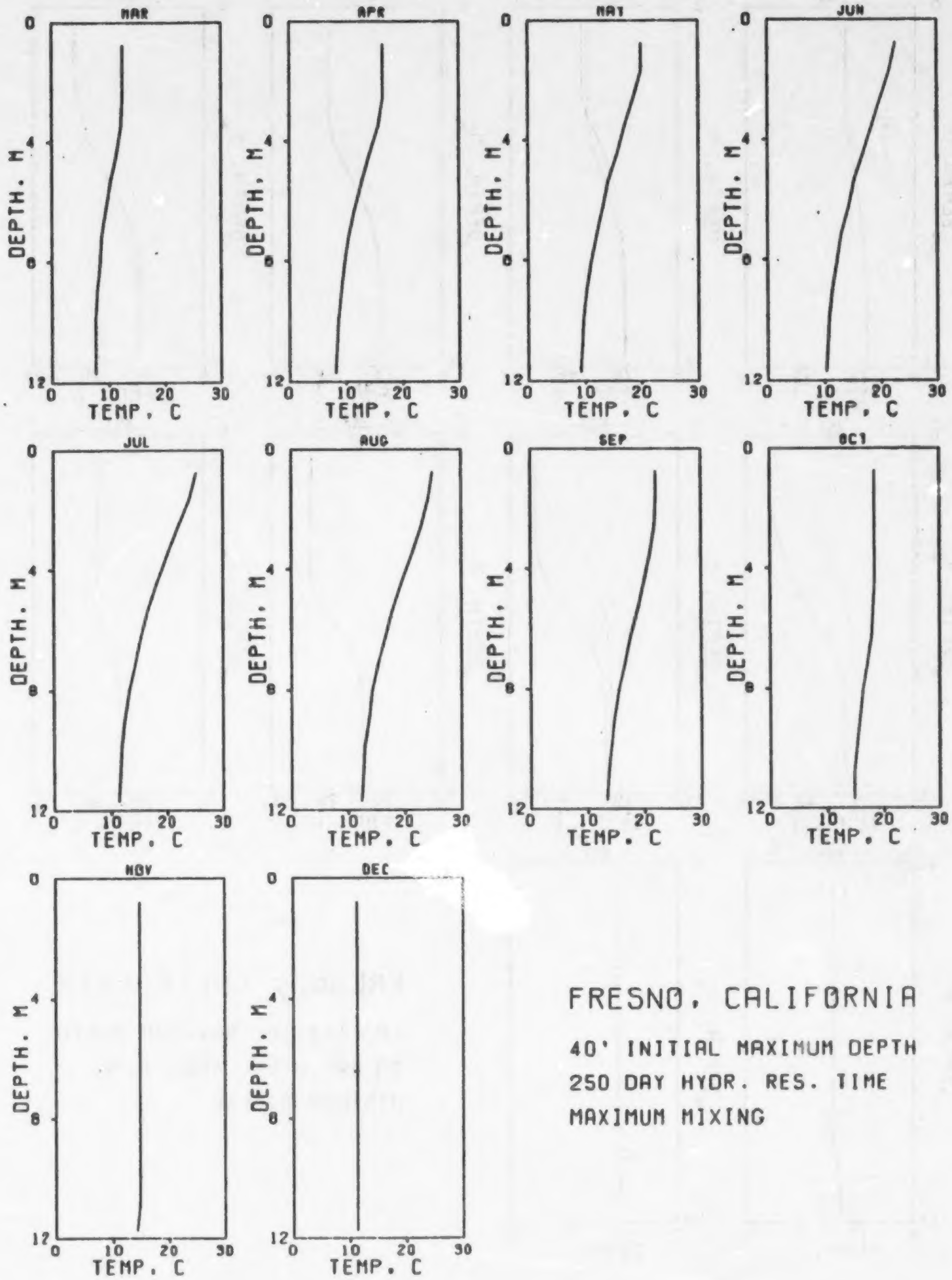
FRESNO, CALIFORNIA

40' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



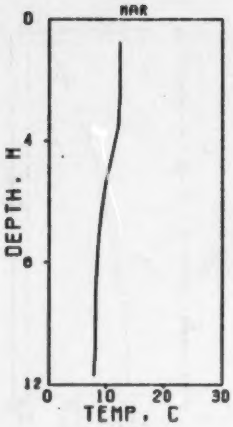
FRESNO, CALIFORNIA

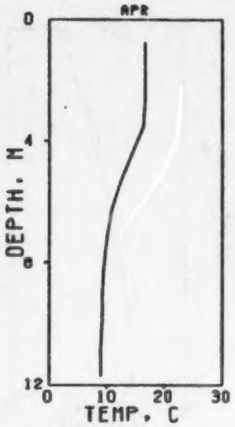
40' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING

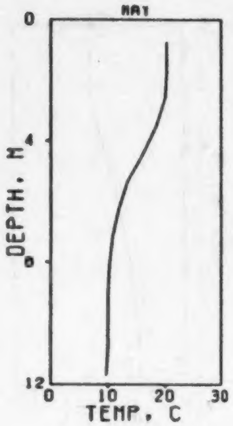


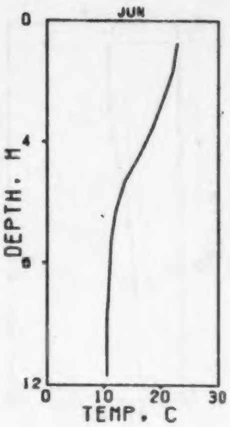
FRESNO, CALIFORNIA

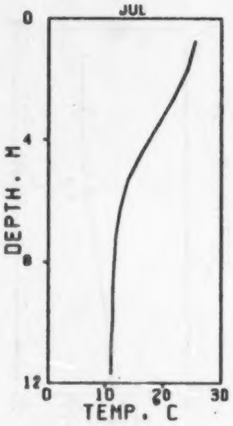
40' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING

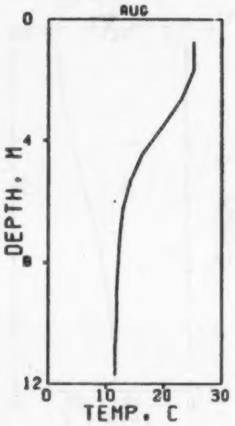


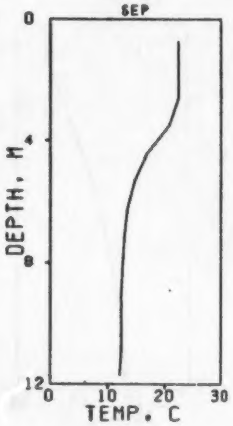


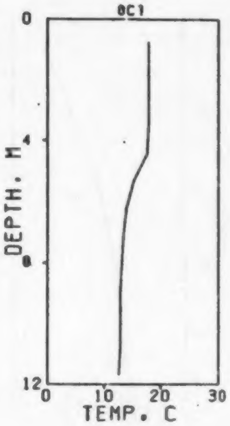


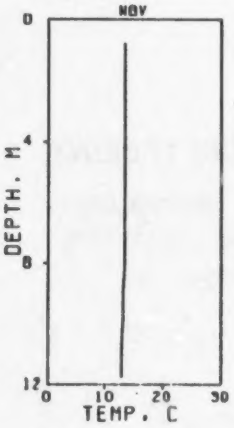


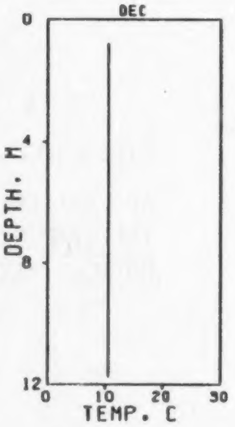






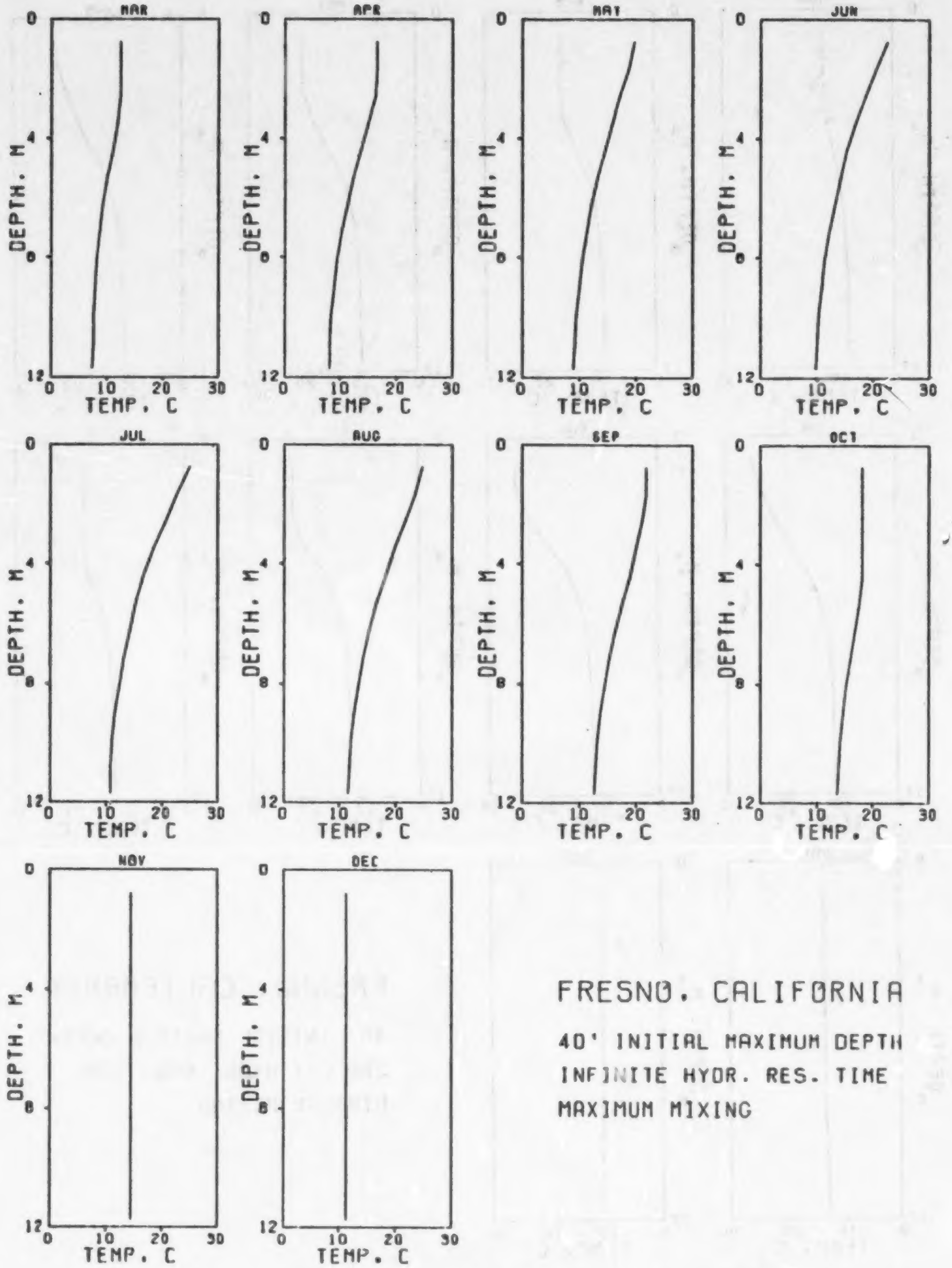






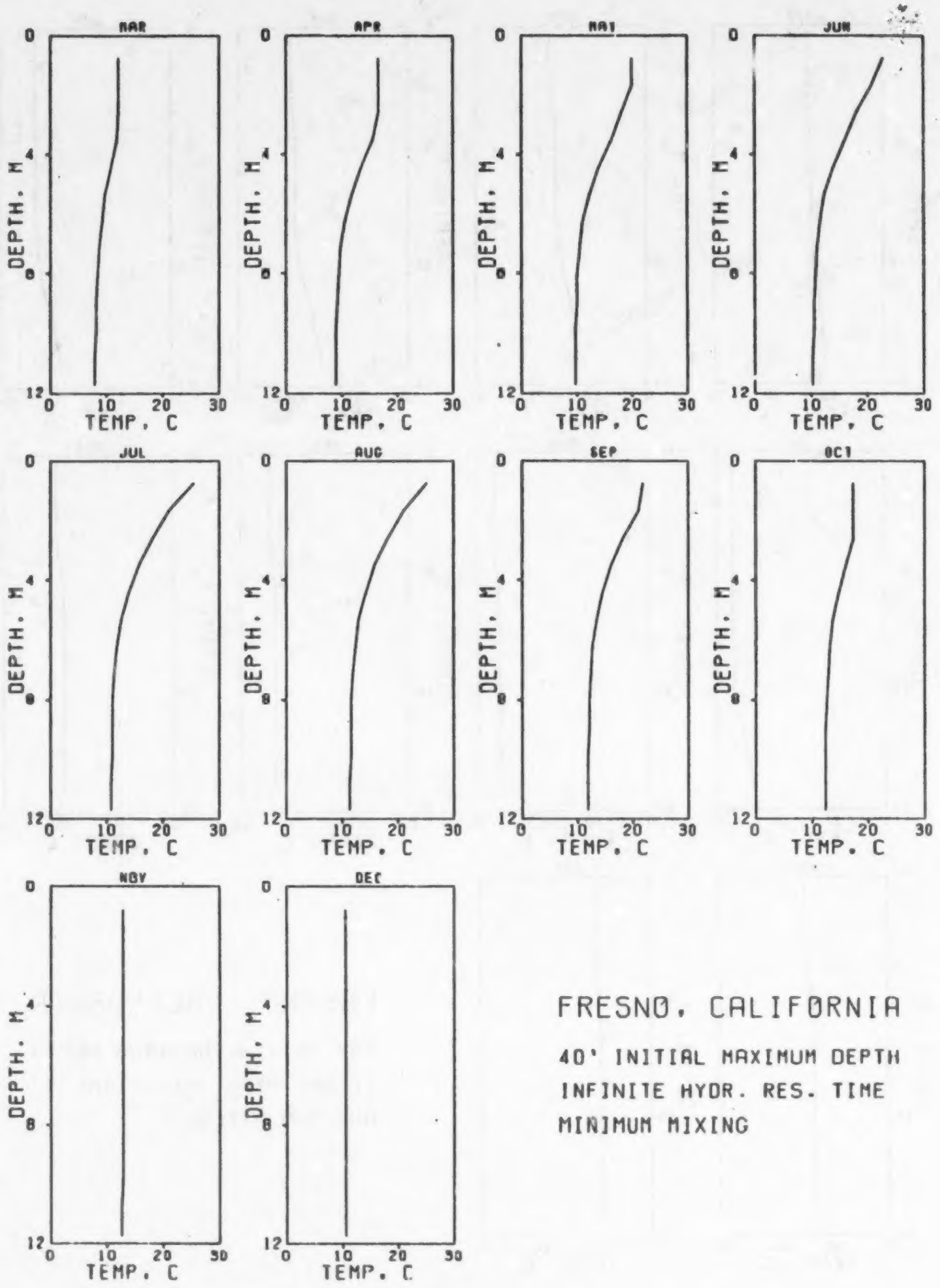
FRESNO, CALIFORNIA

40' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



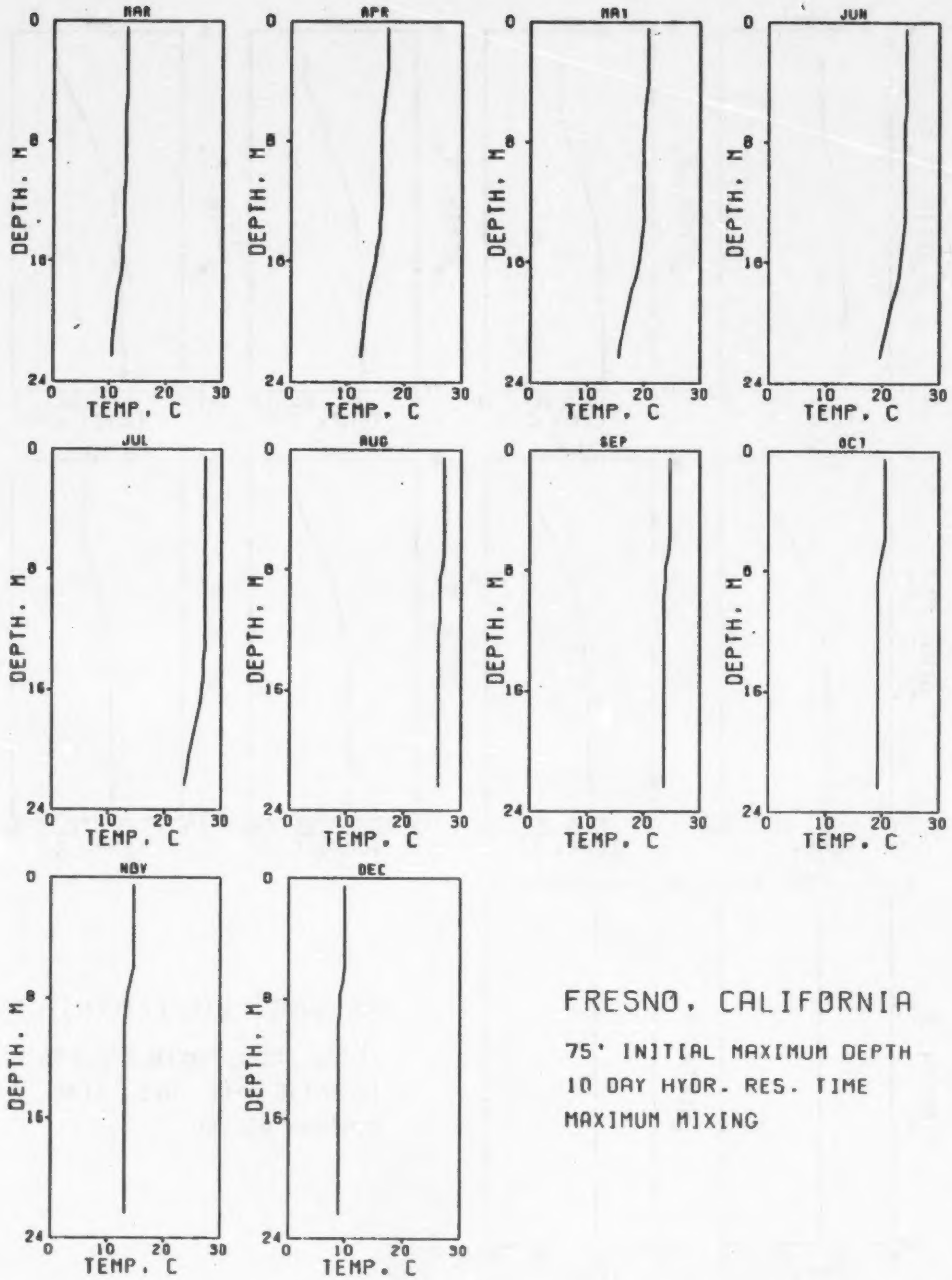
FRESNO. CALIFORNIA

40' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



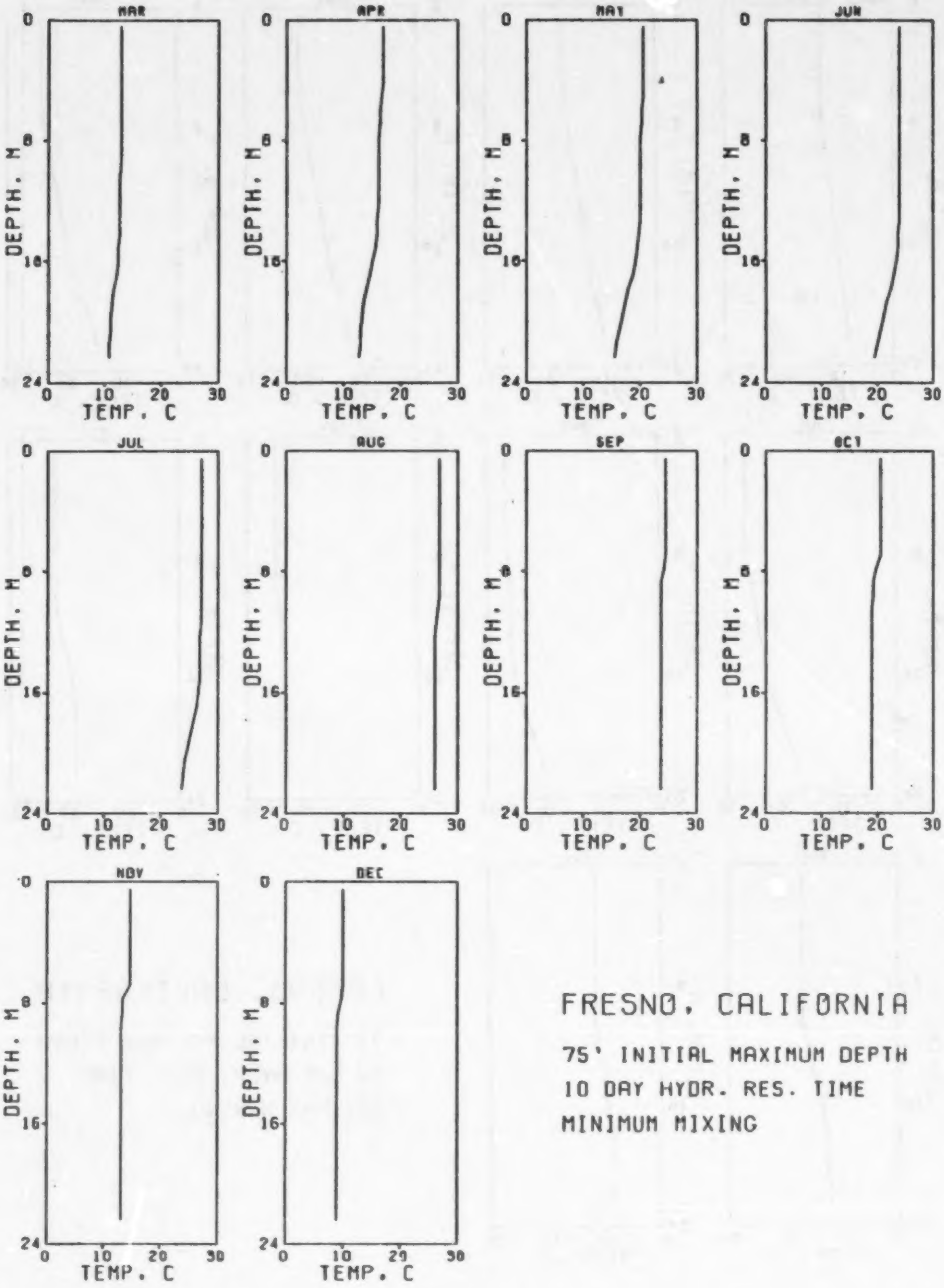
FRESNO, CALIFORNIA

40' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



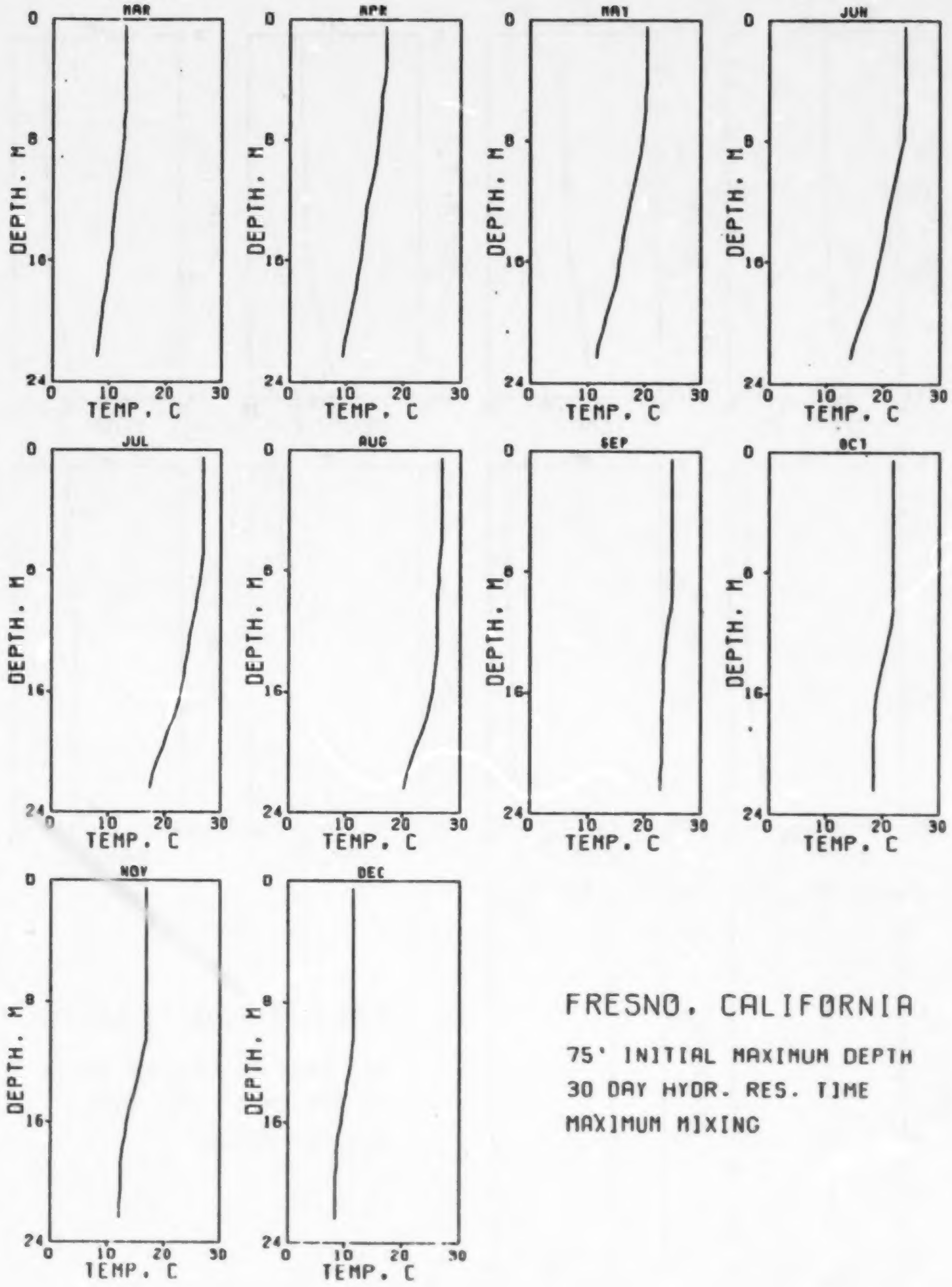
FRESNO, CALIFORNIA

75' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



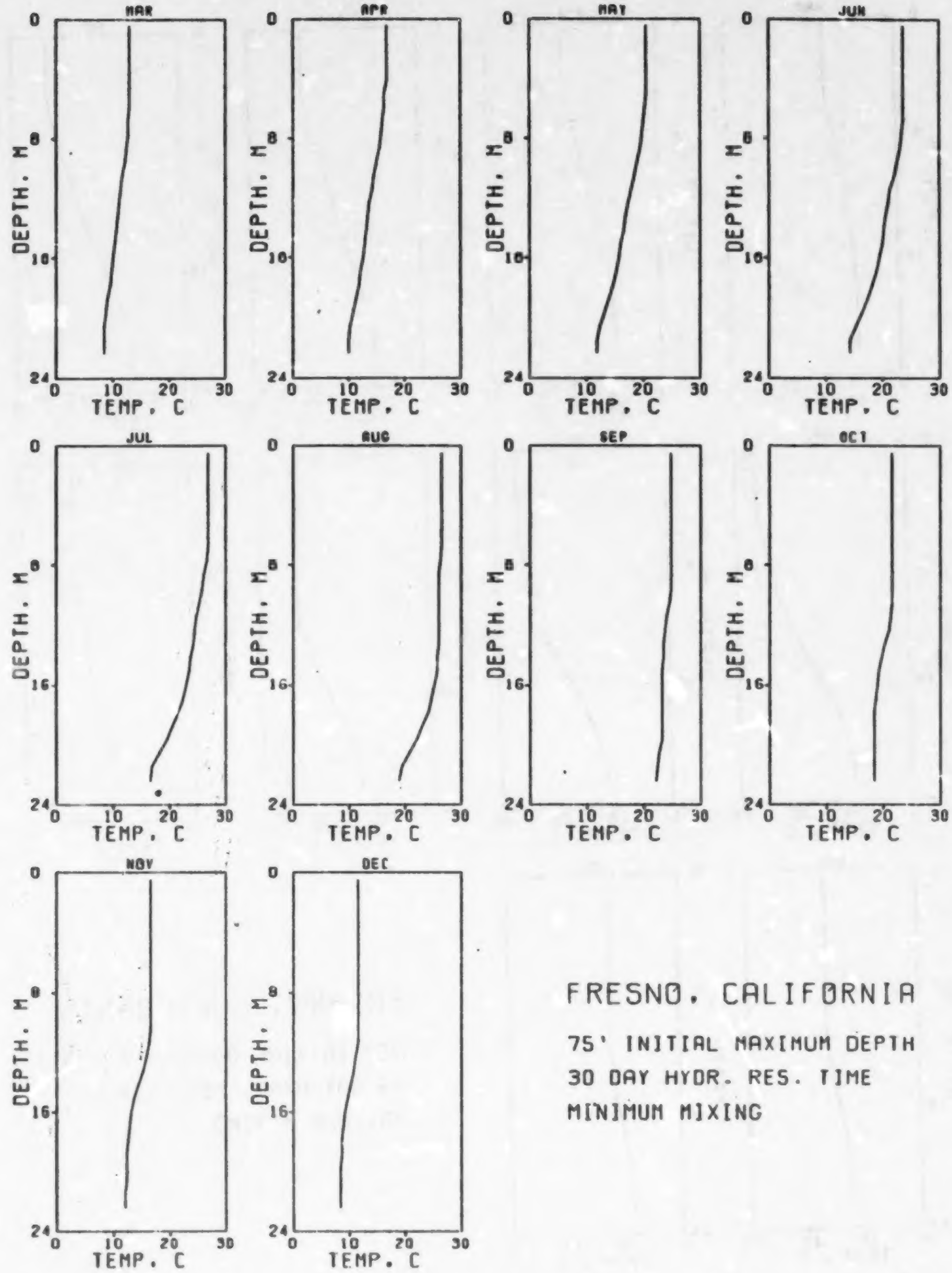
FRESNO, CALIFORNIA

75' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



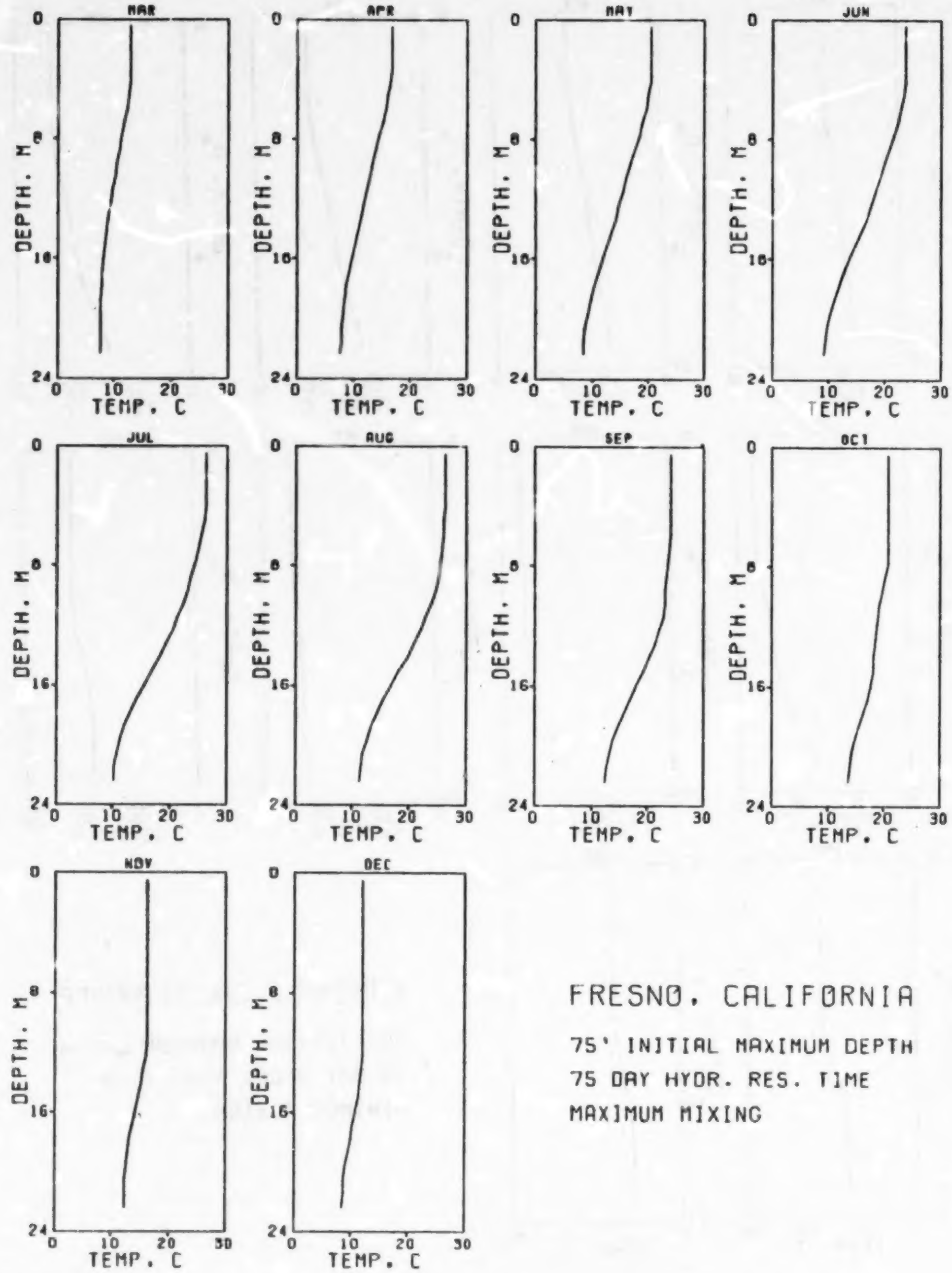
FRESNO, CALIFORNIA

75' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



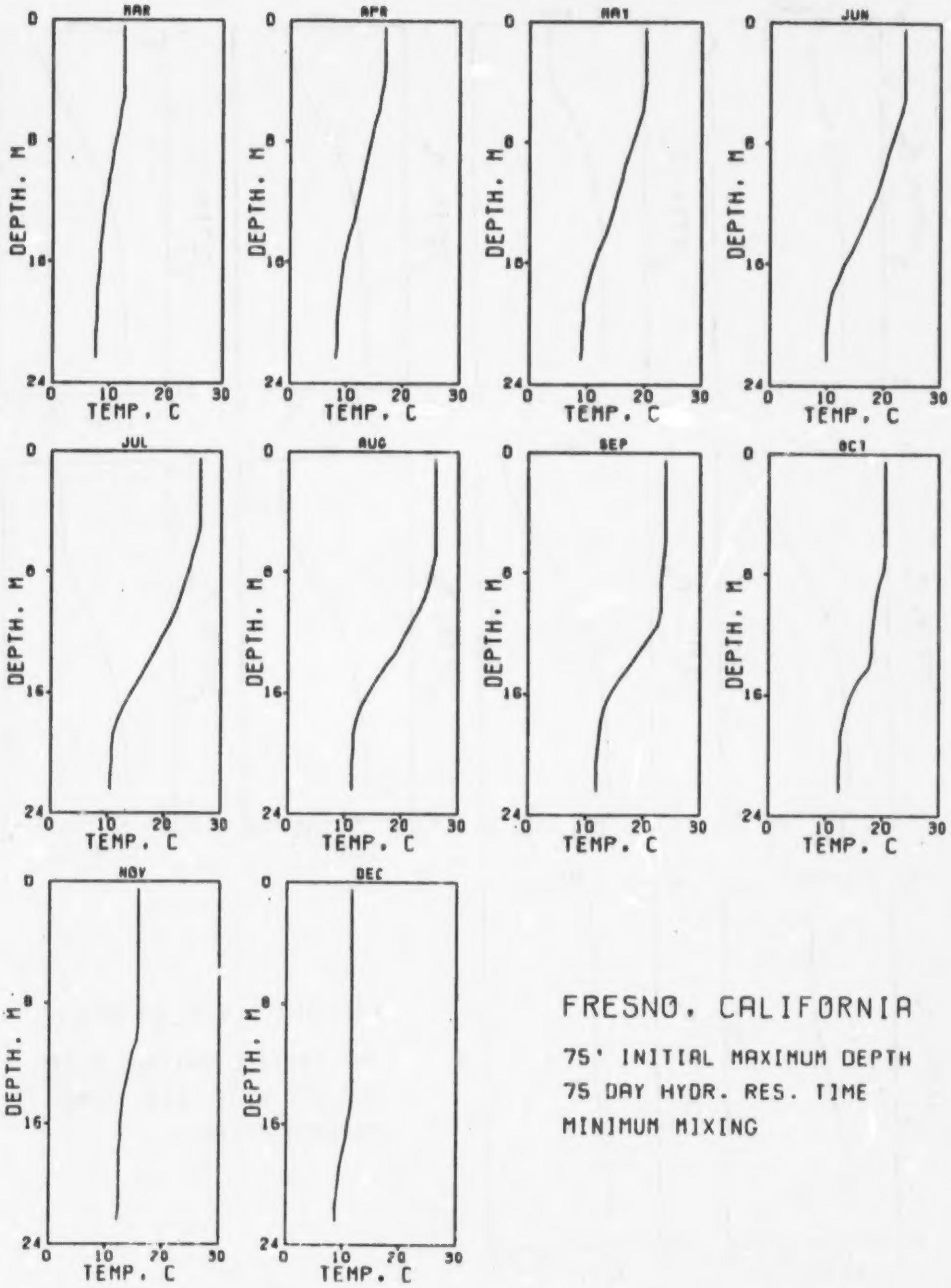
FRESNO, CALIFORNIA

75' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



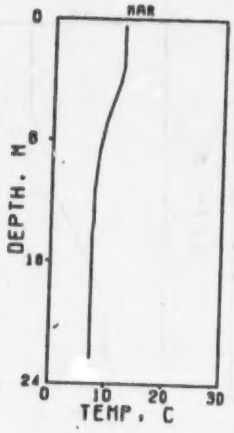
FRESNO, CALIFORNIA

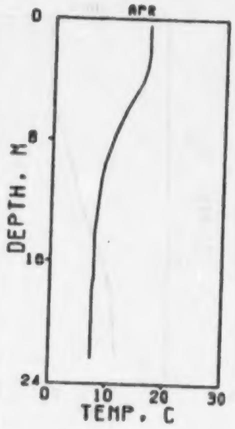
75' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING

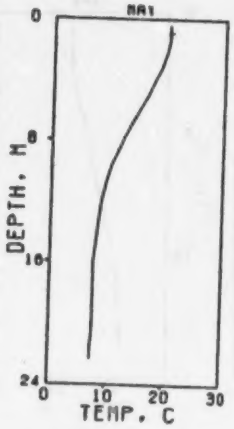


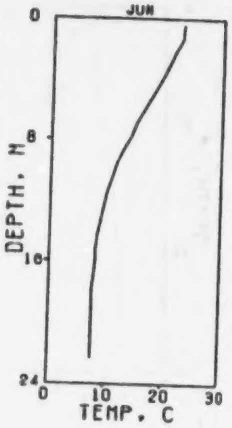
FRESNO, CALIFORNIA

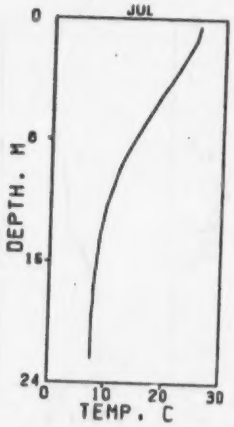
75' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING

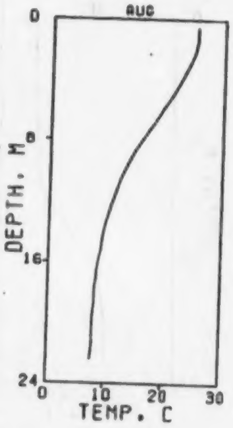


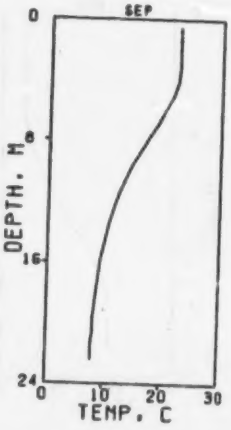


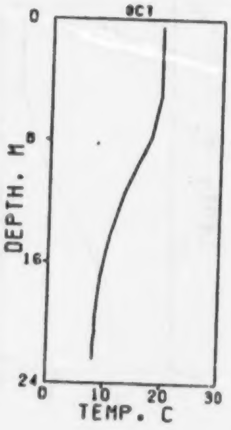


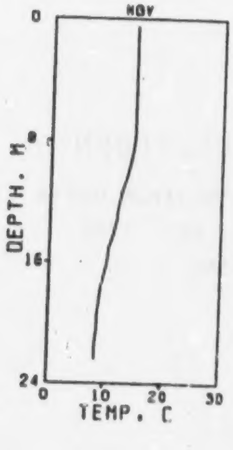


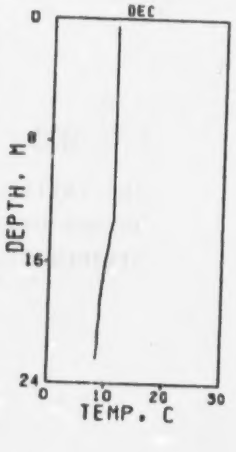






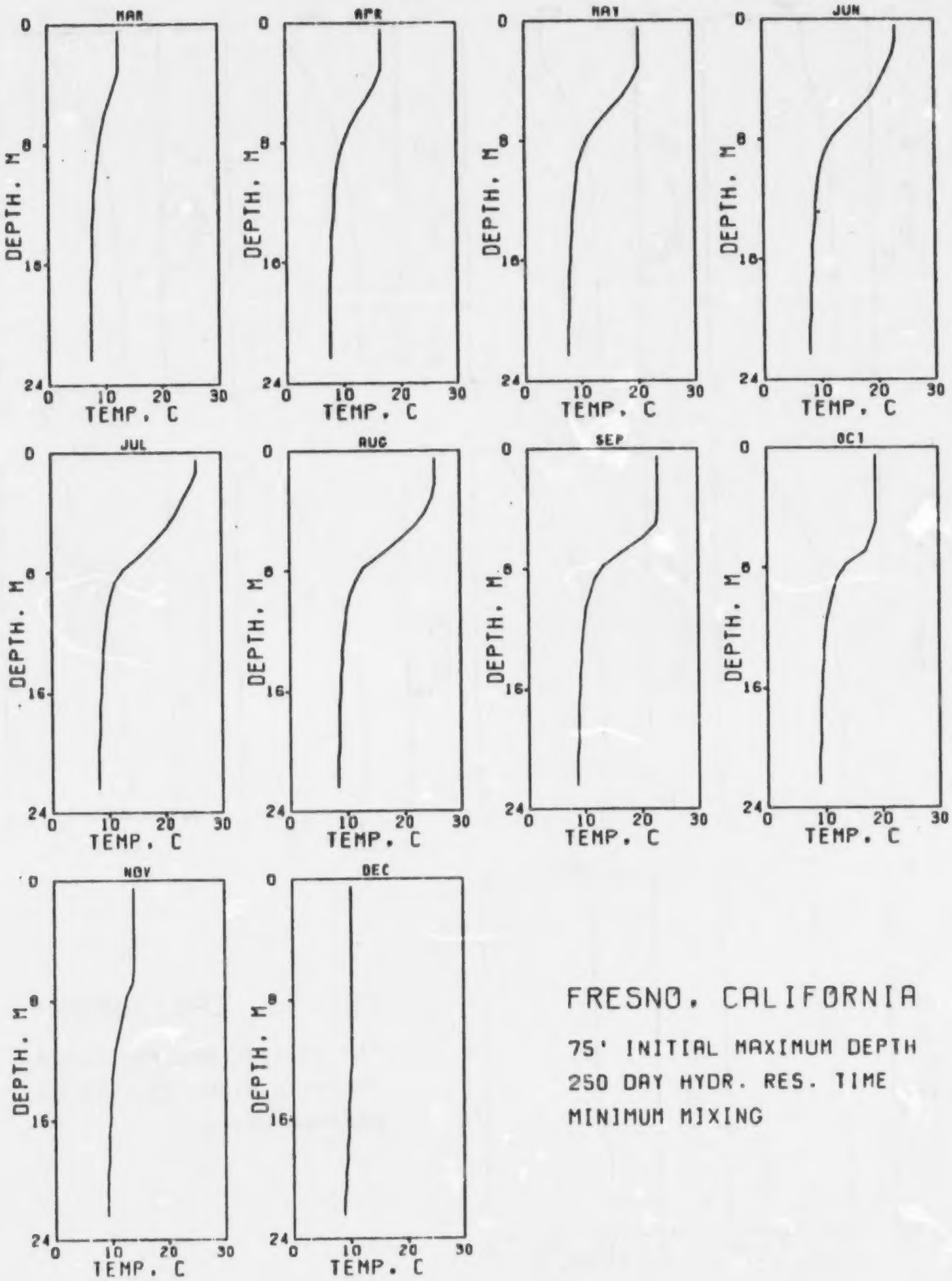






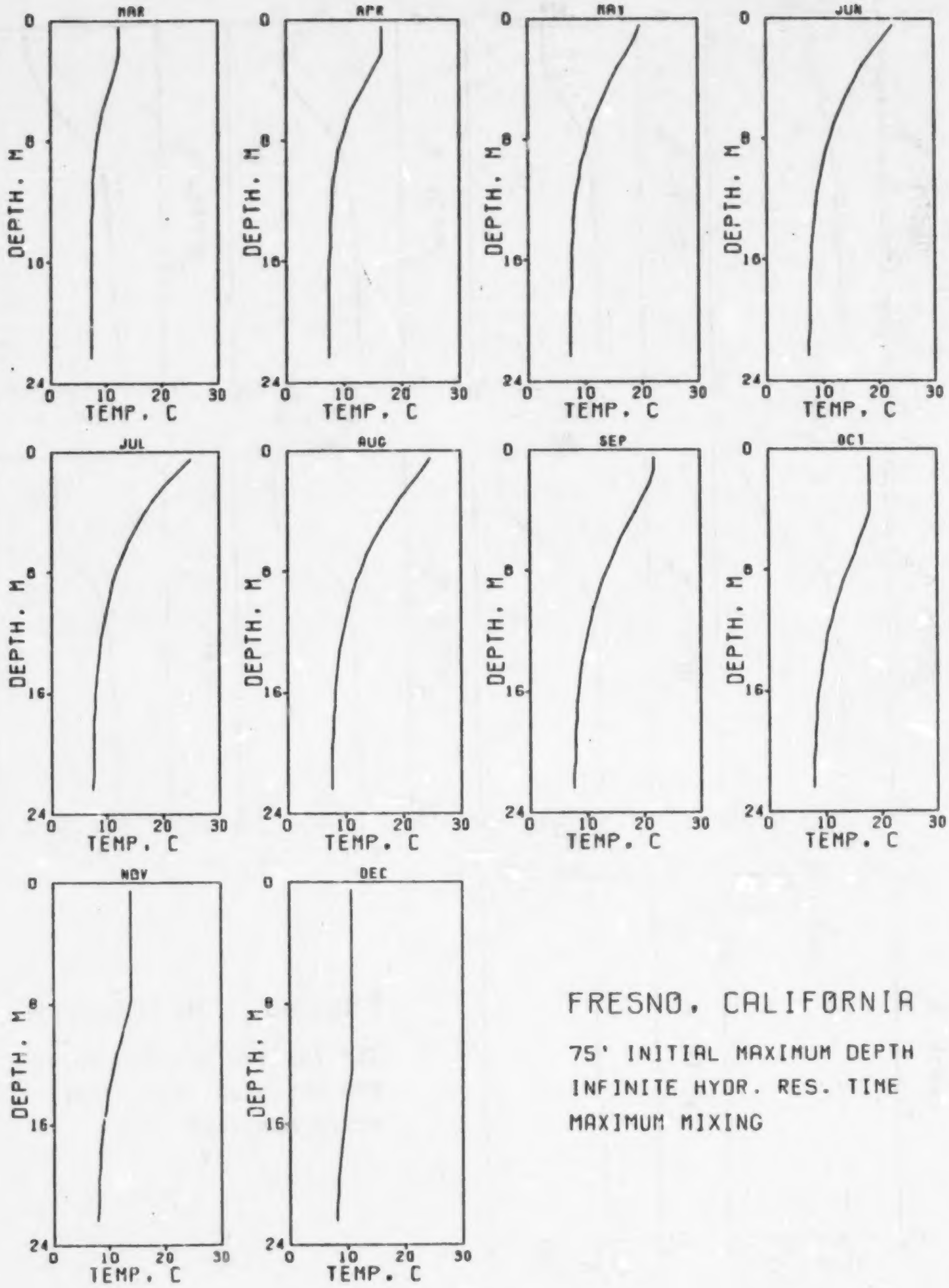
FRESNO, CALIFORNIA

75' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



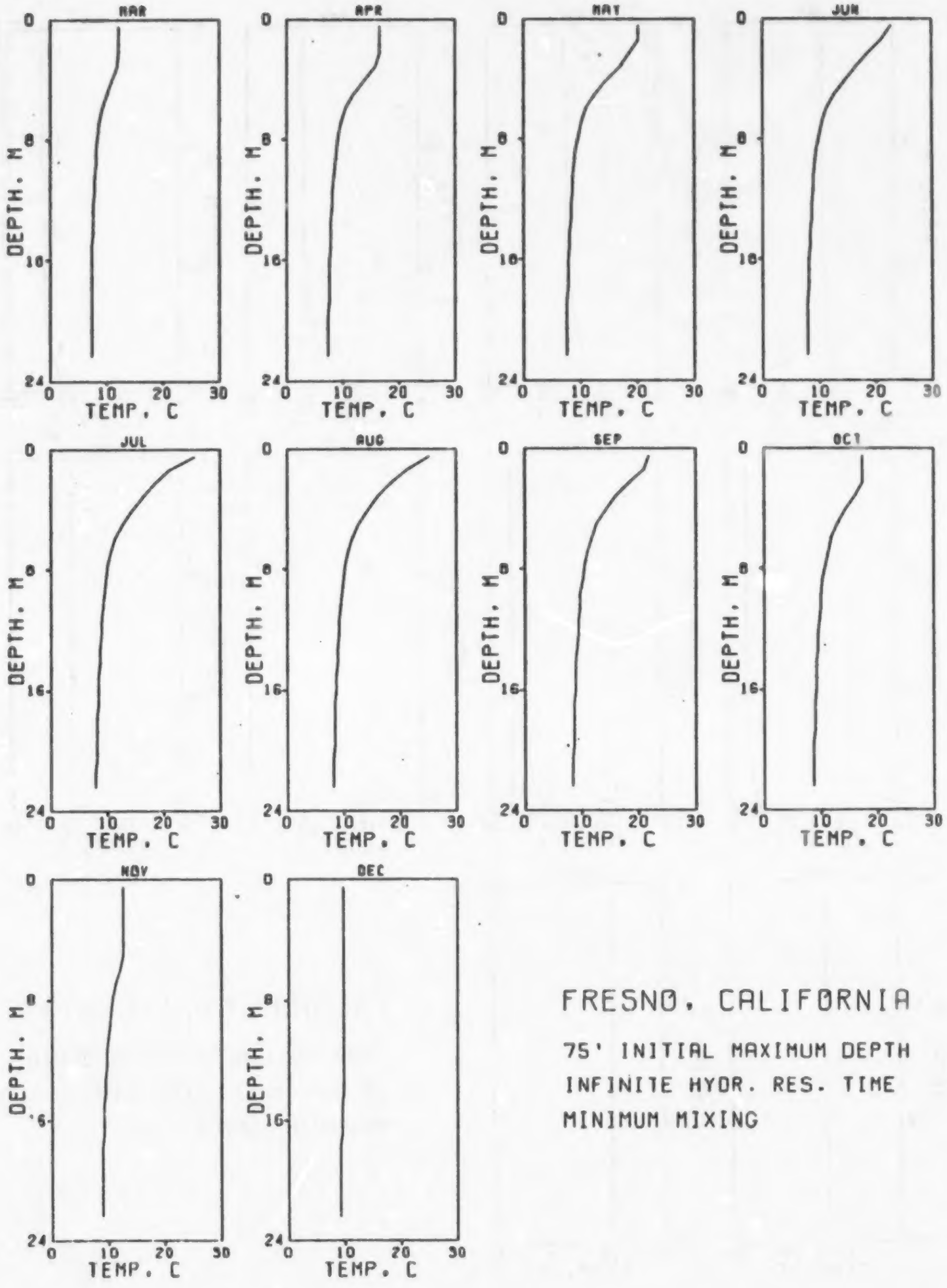
FRESNO, CALIFORNIA

75' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



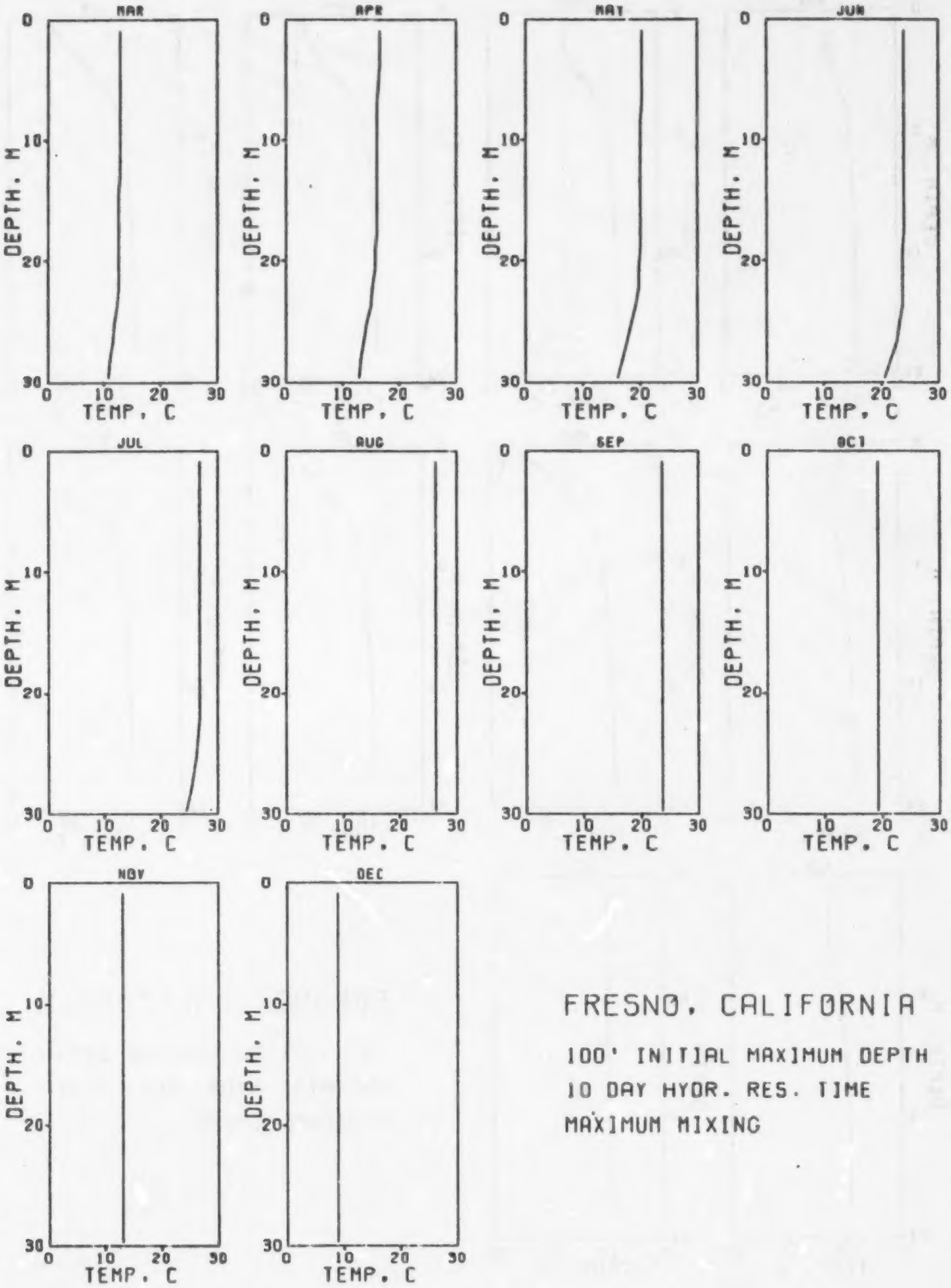
FRESNO, CALIFORNIA

75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



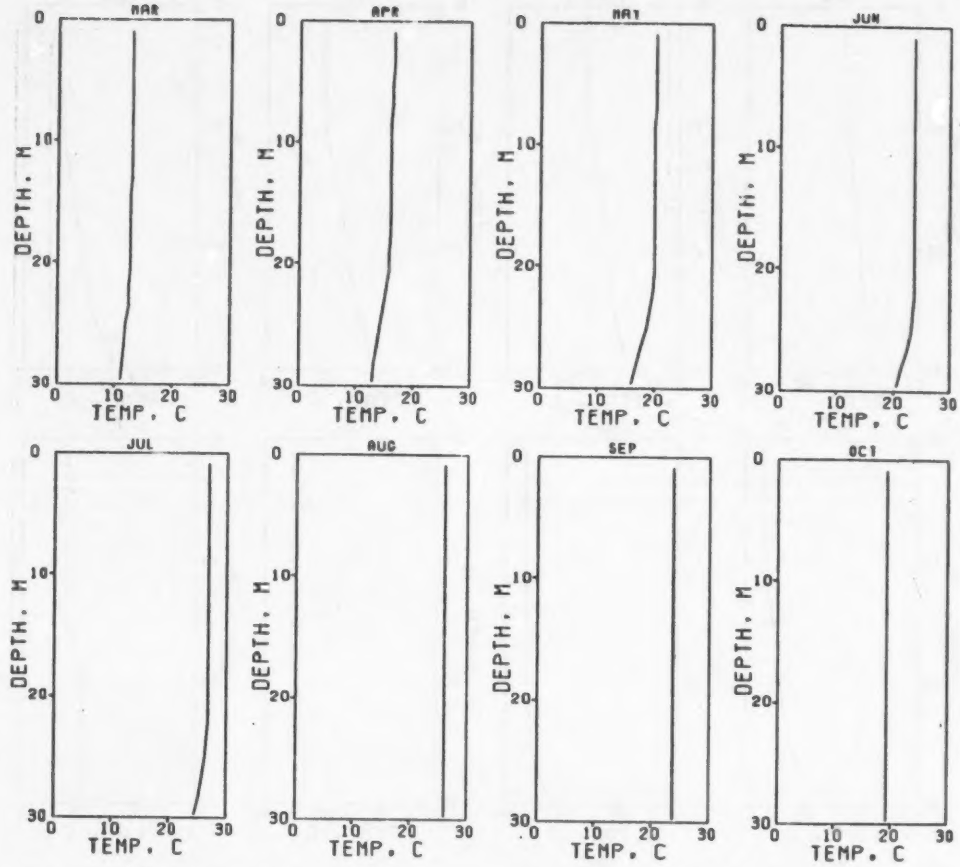
FRESNO, CALIFORNIA

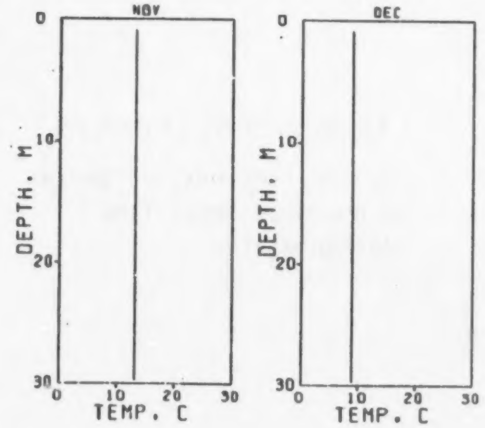
75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



FRESNO, CALIFORNIA

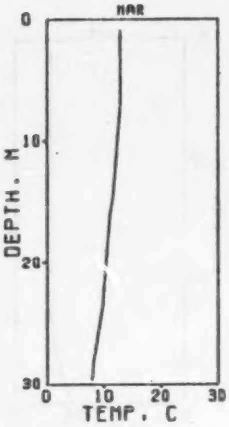
100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING

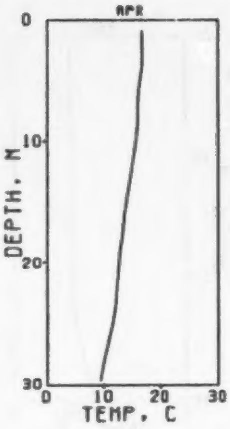


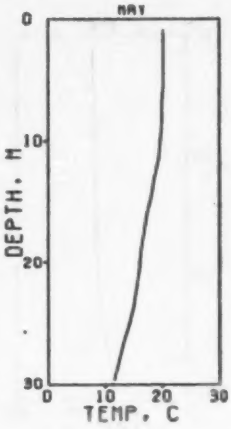


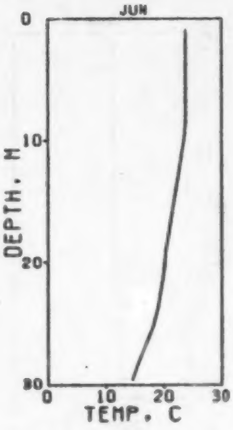
FRESNO, CALIFORNIA

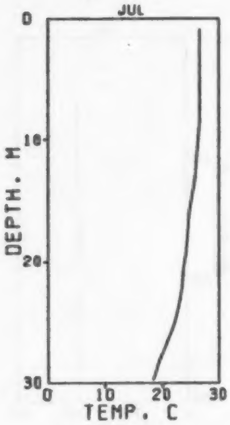
100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING

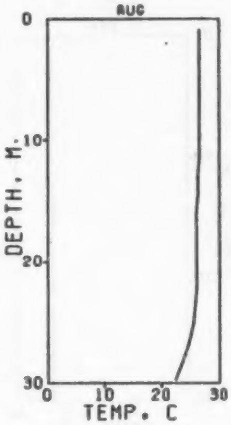


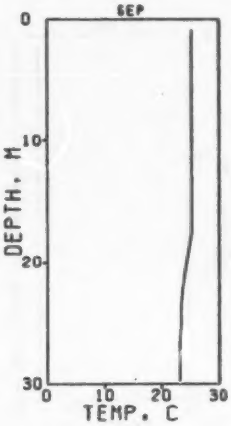


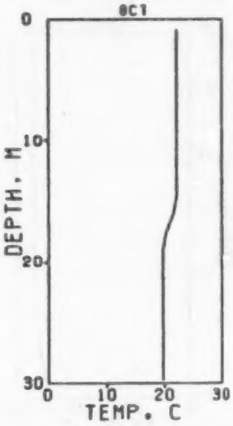


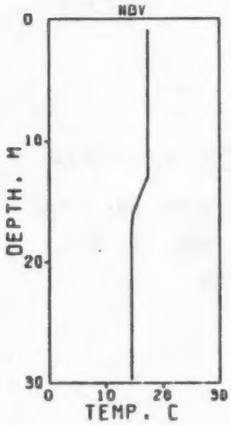


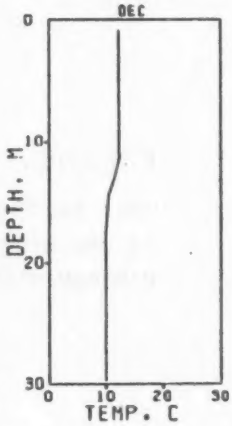






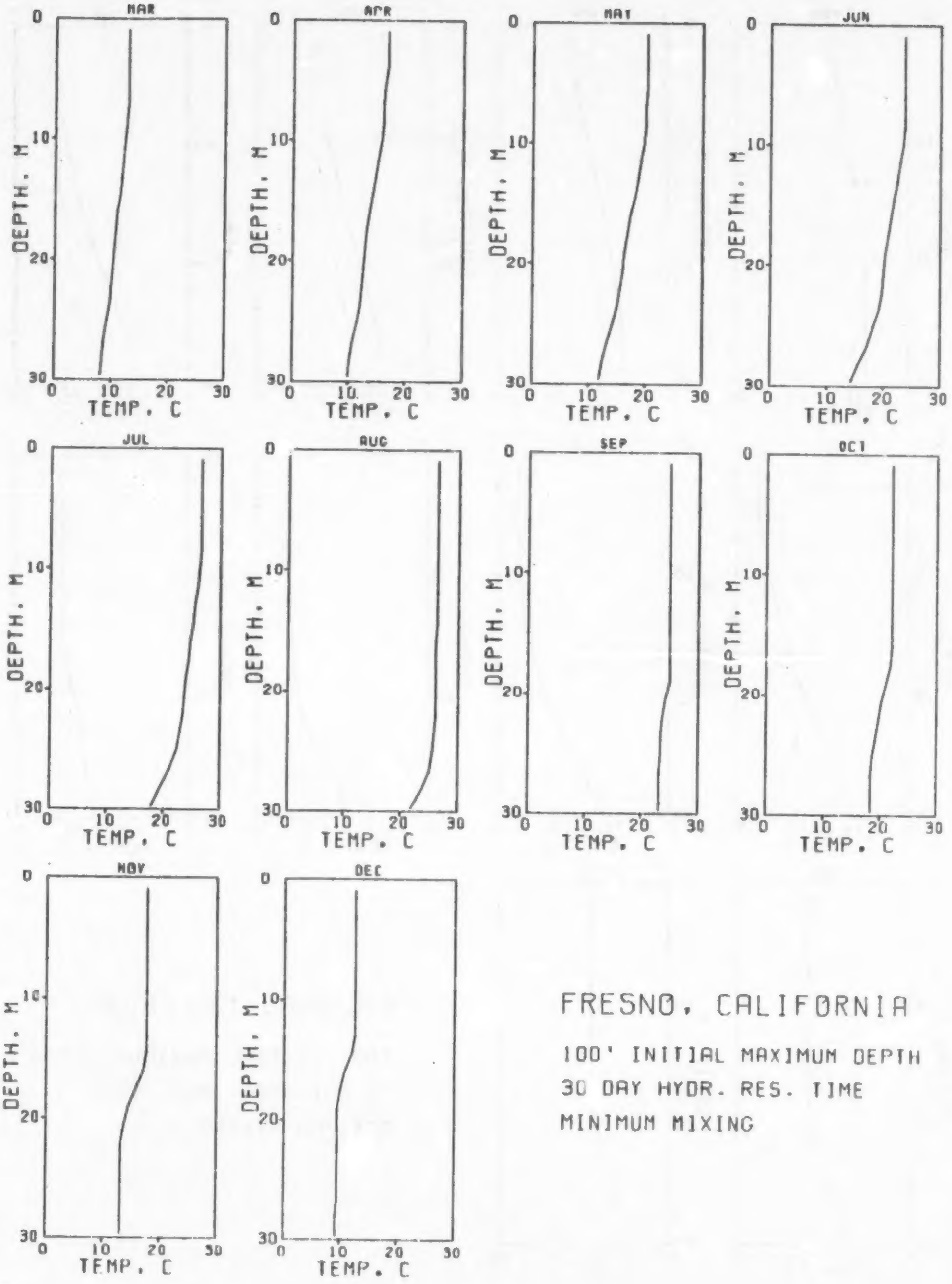






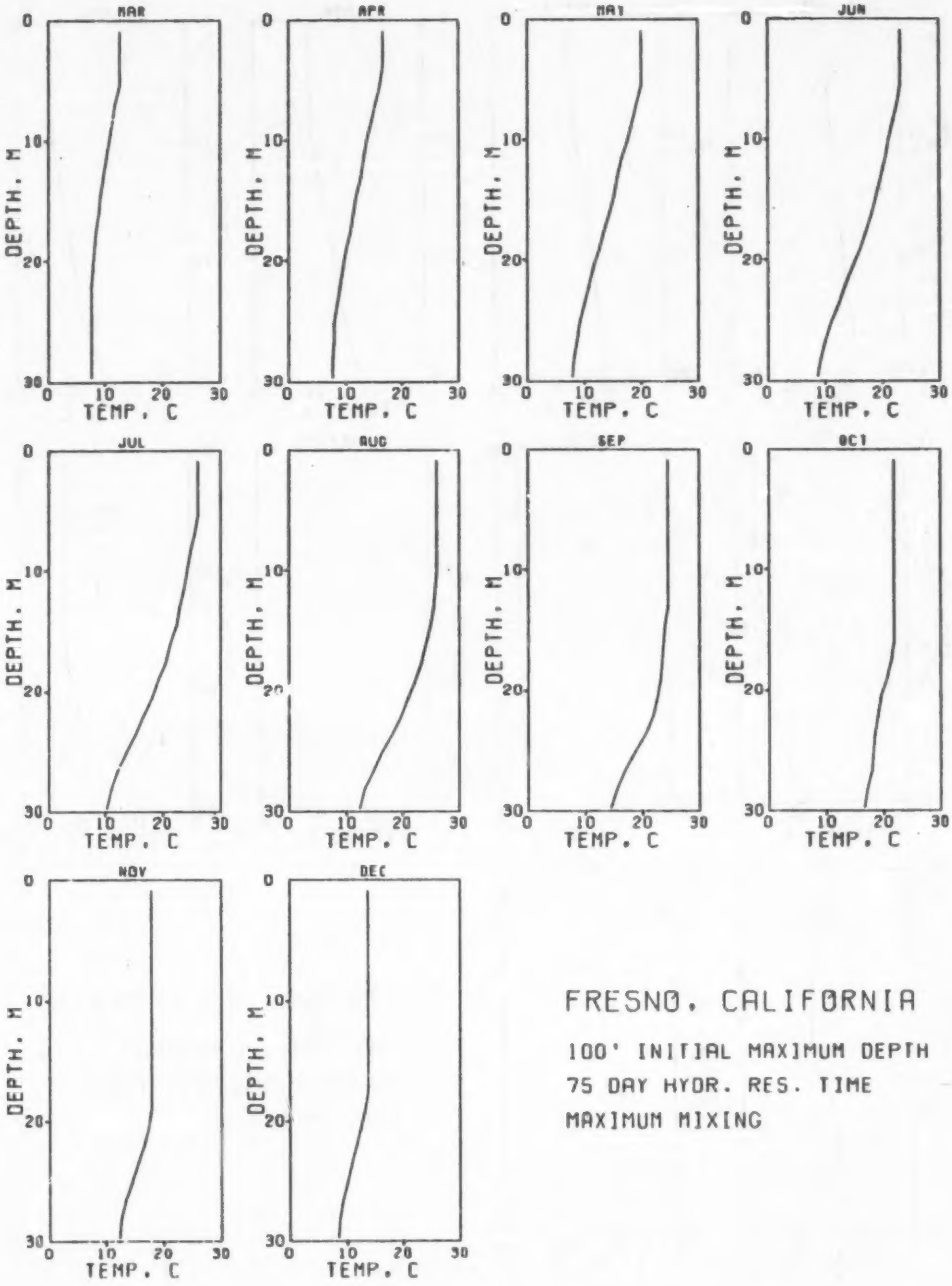
FRESNO, CALIFORNIA

100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



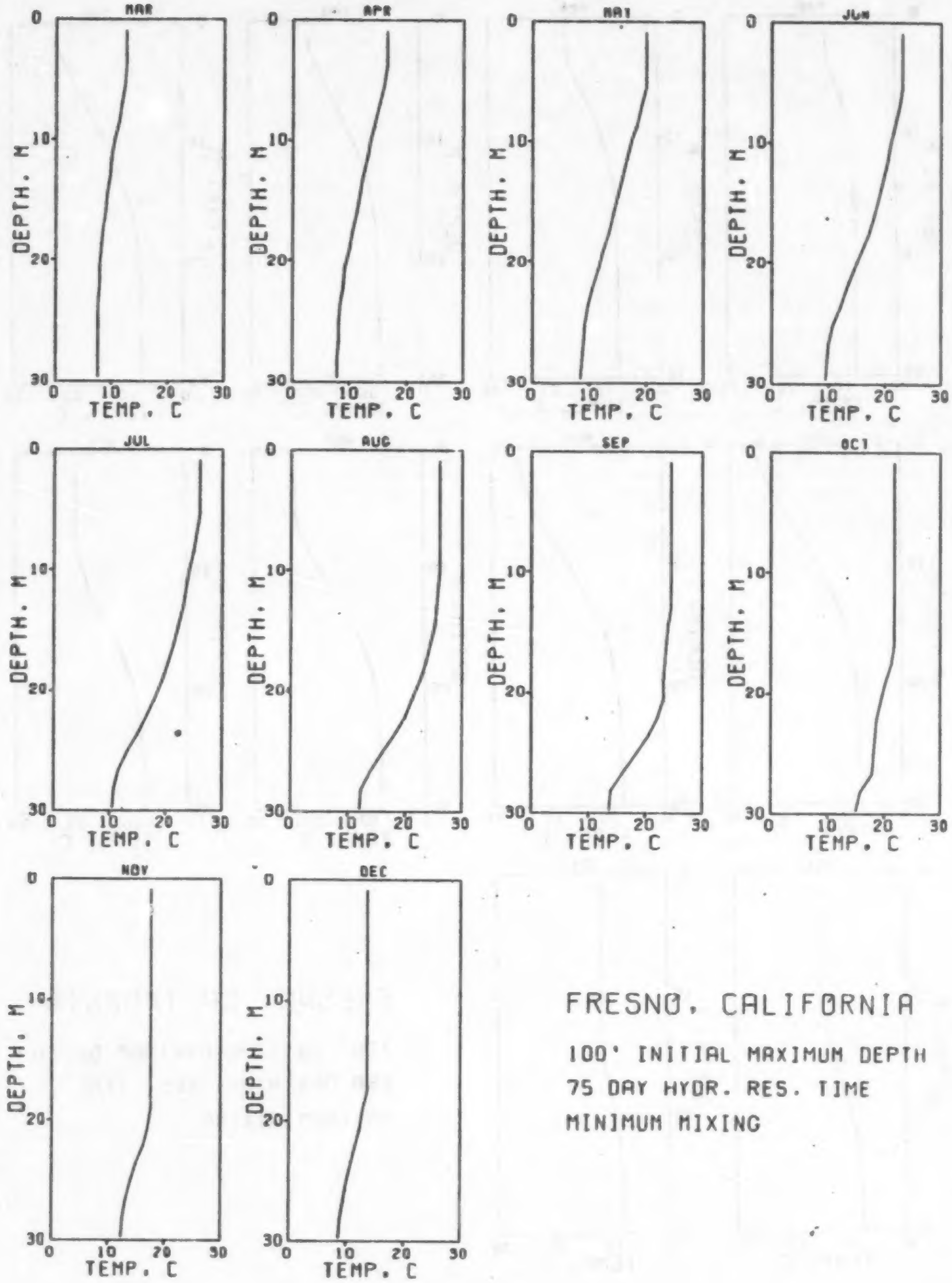
FRESNO, CALIFORNIA

100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



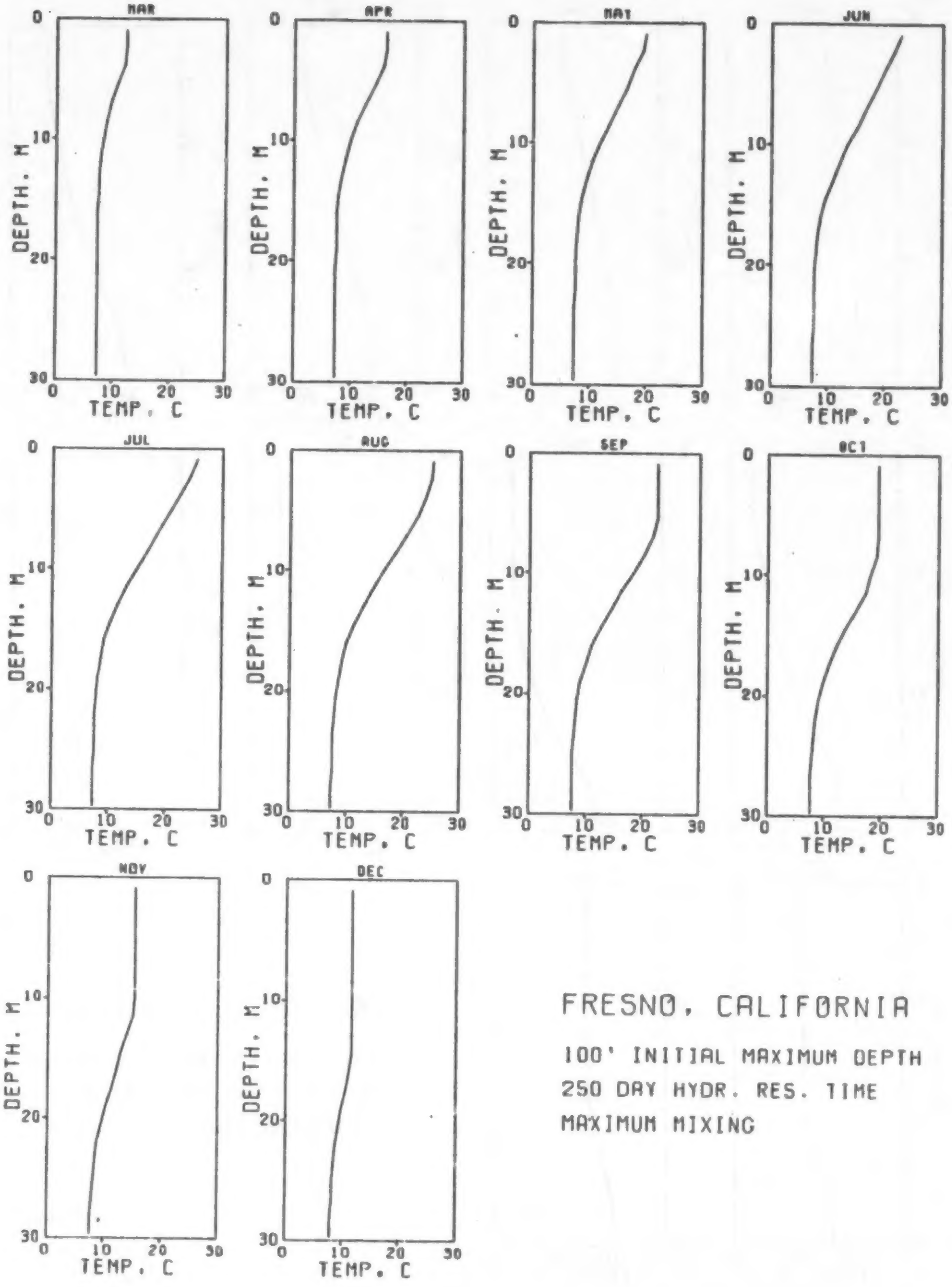
FRESNO, CALIFORNIA

100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



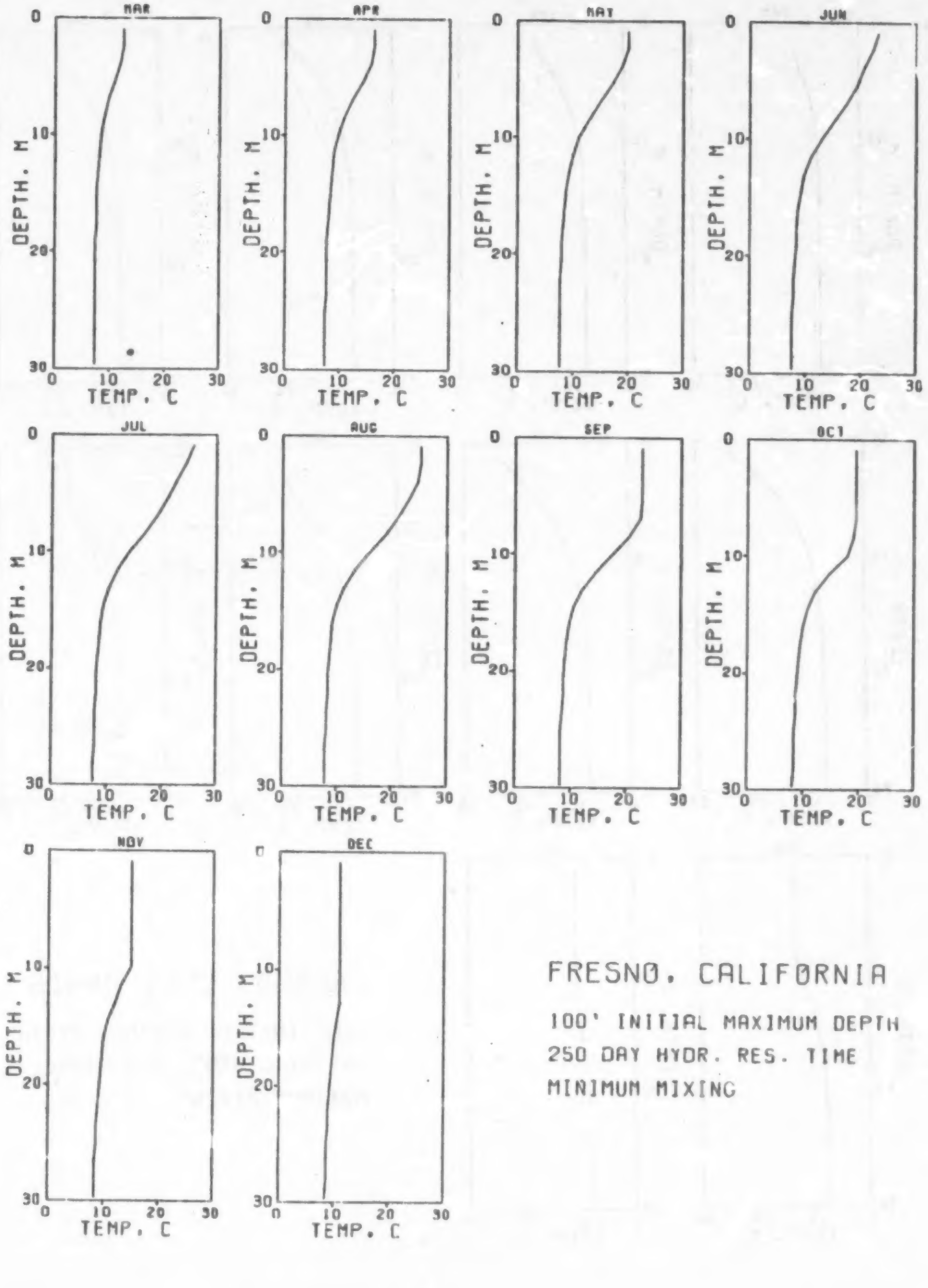
FRESNO, CALIFORNIA

100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



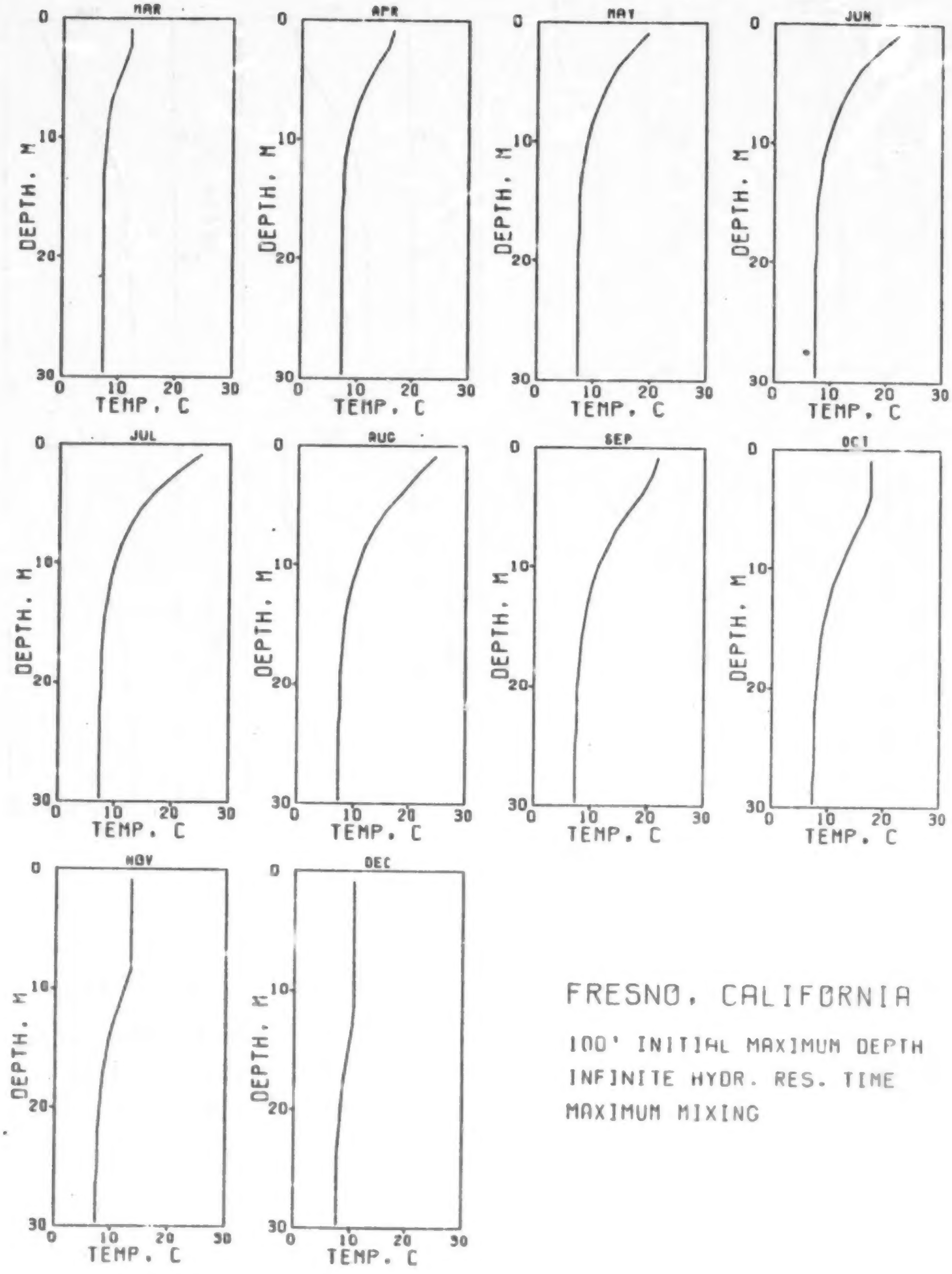
FRESNO, CALIFORNIA

100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



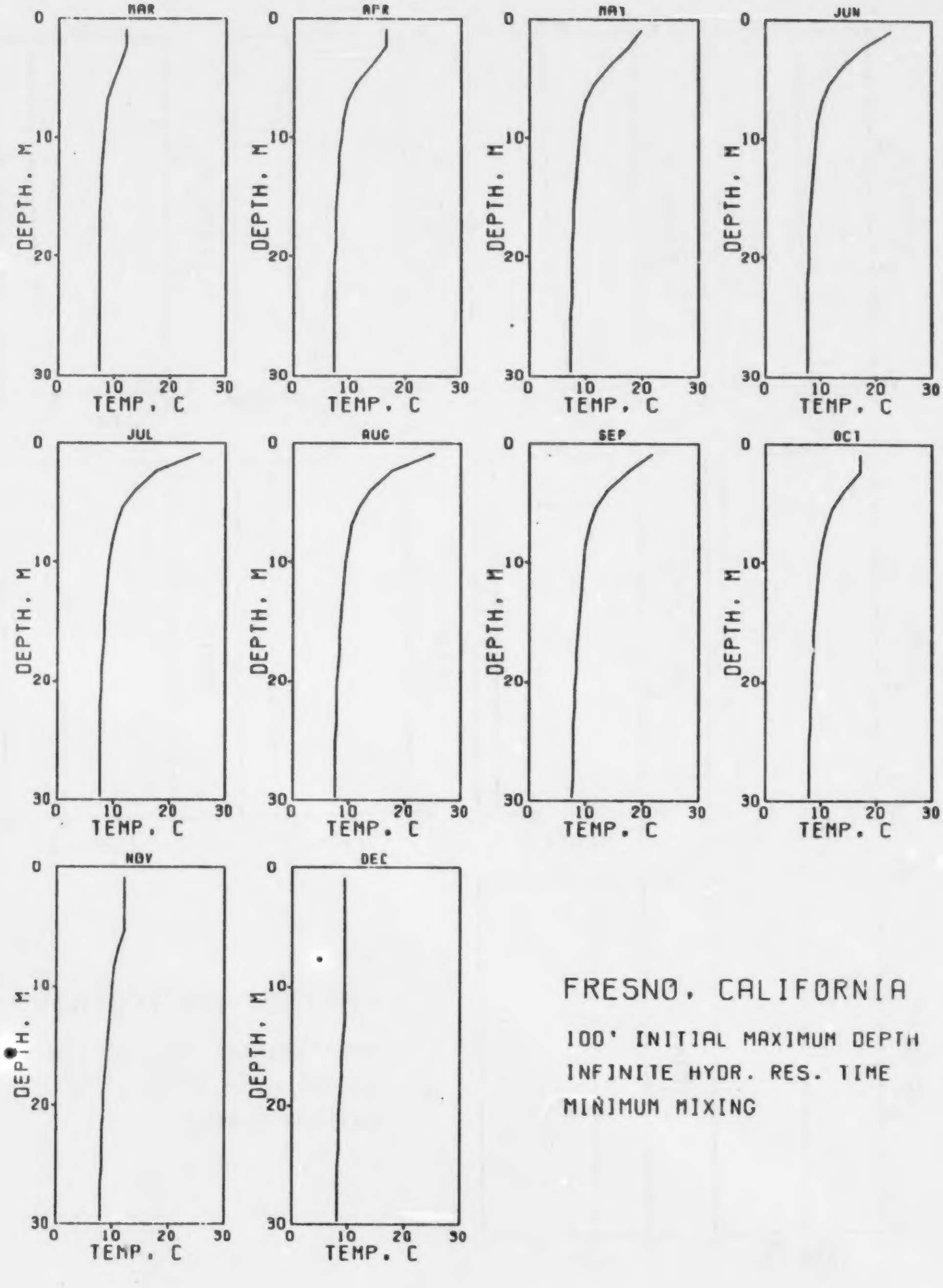
FRESNO, CALIFORNIA

100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



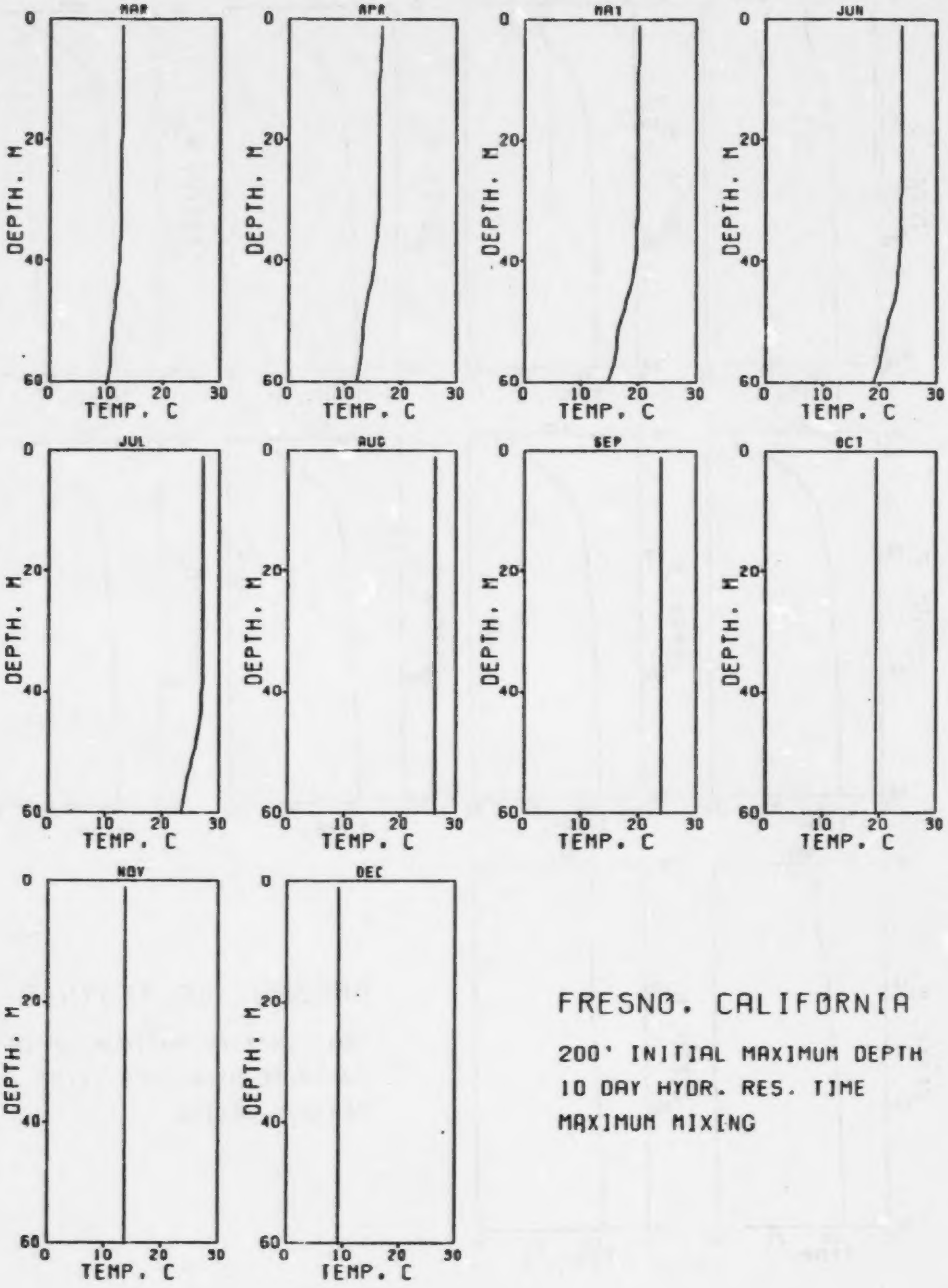
FRESNO, CALIFORNIA

100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



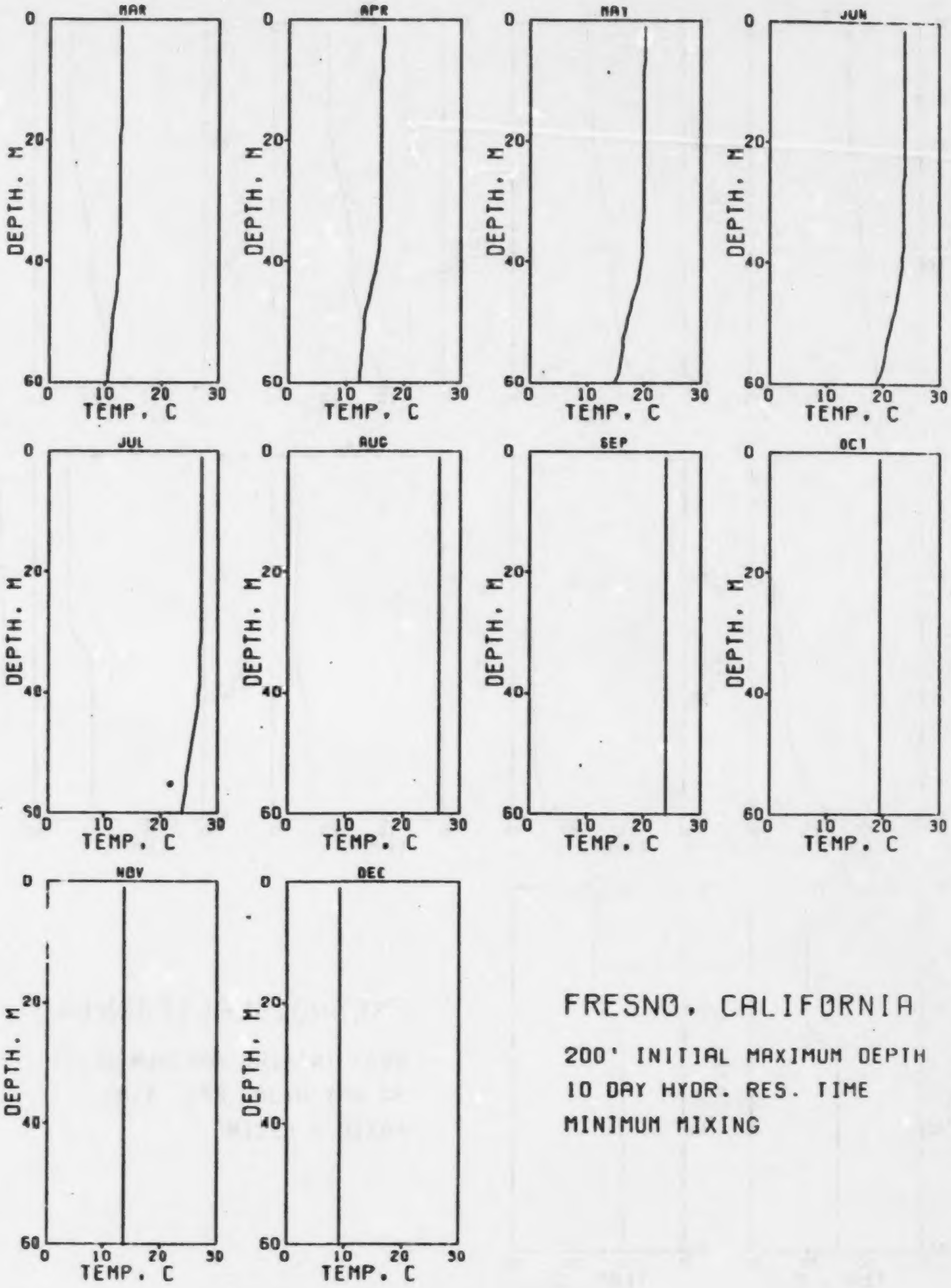
FRESNO, CALIFORNIA

100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



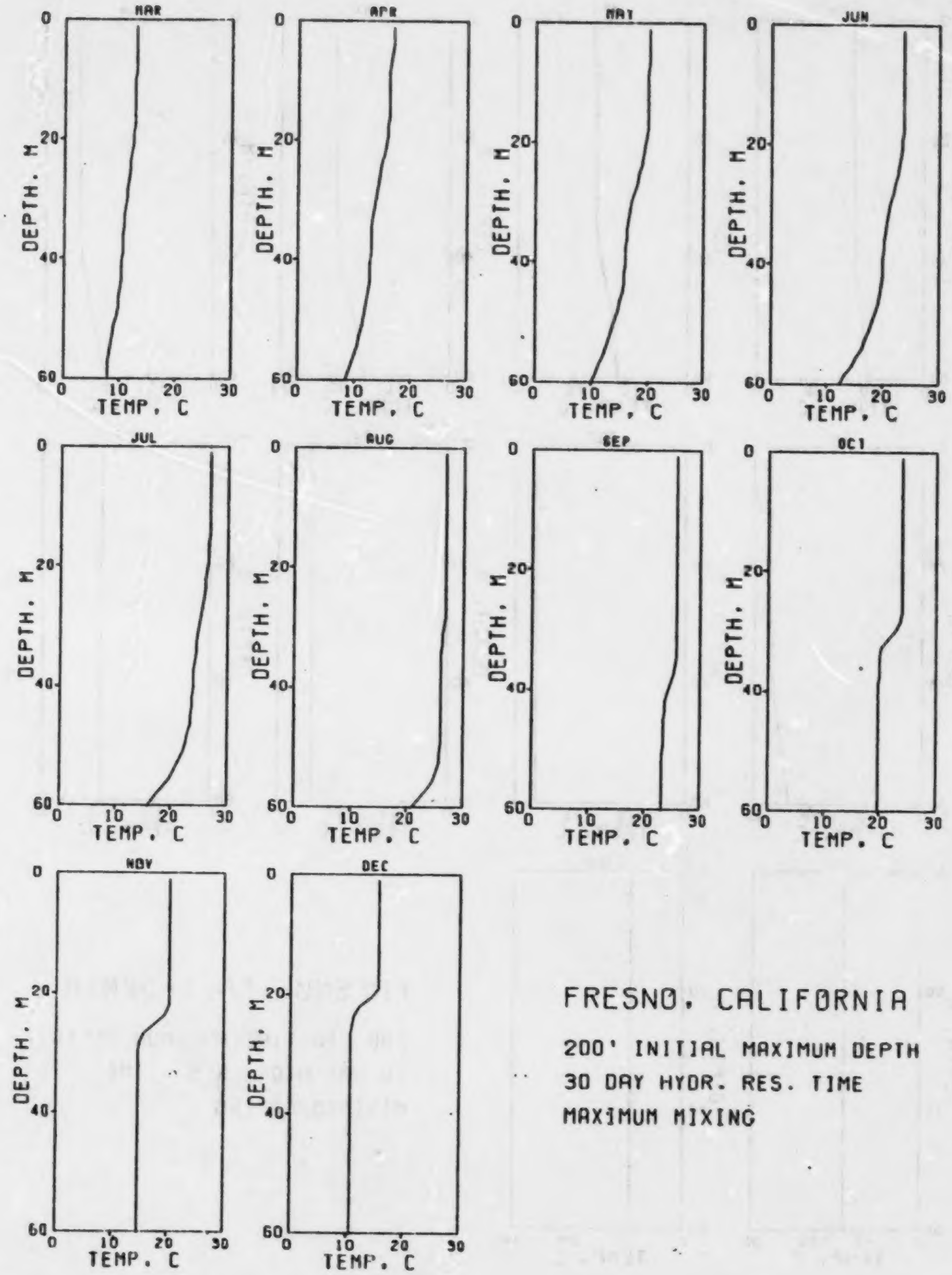
FRESNO, CALIFORNIA

200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



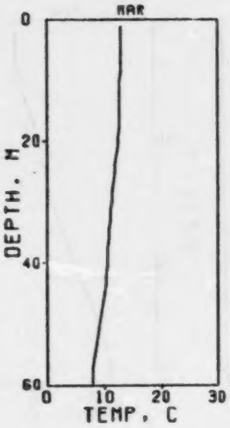
FRESNO, CALIFORNIA

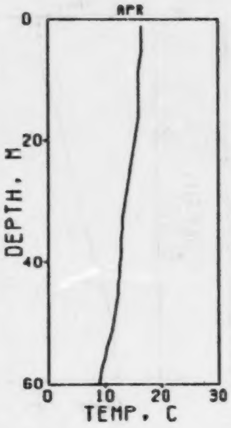
200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING

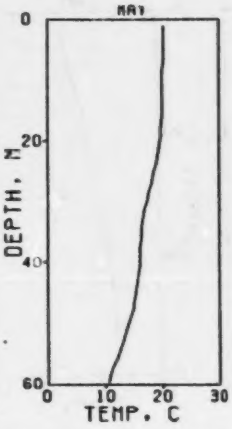


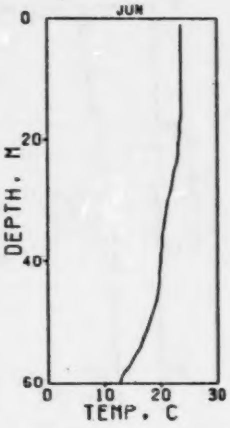
FRESNO, CALIFORNIA

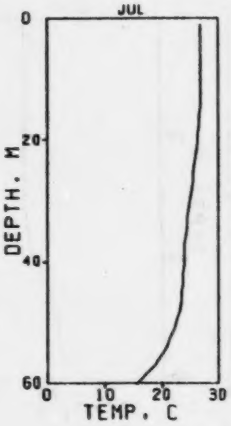
200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING

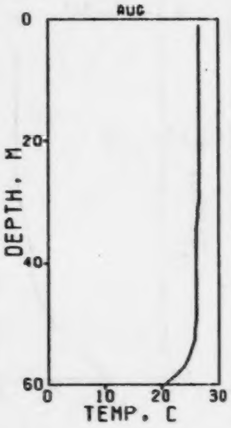


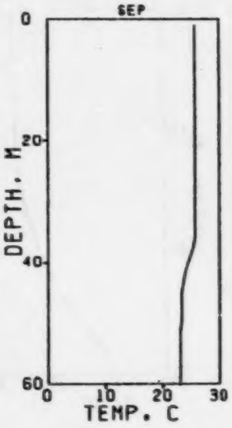


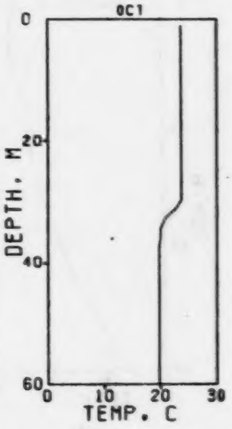


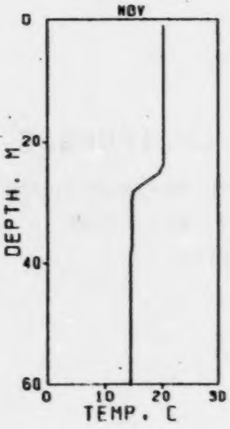


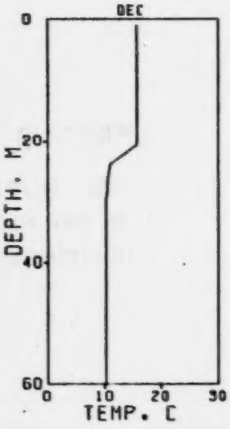






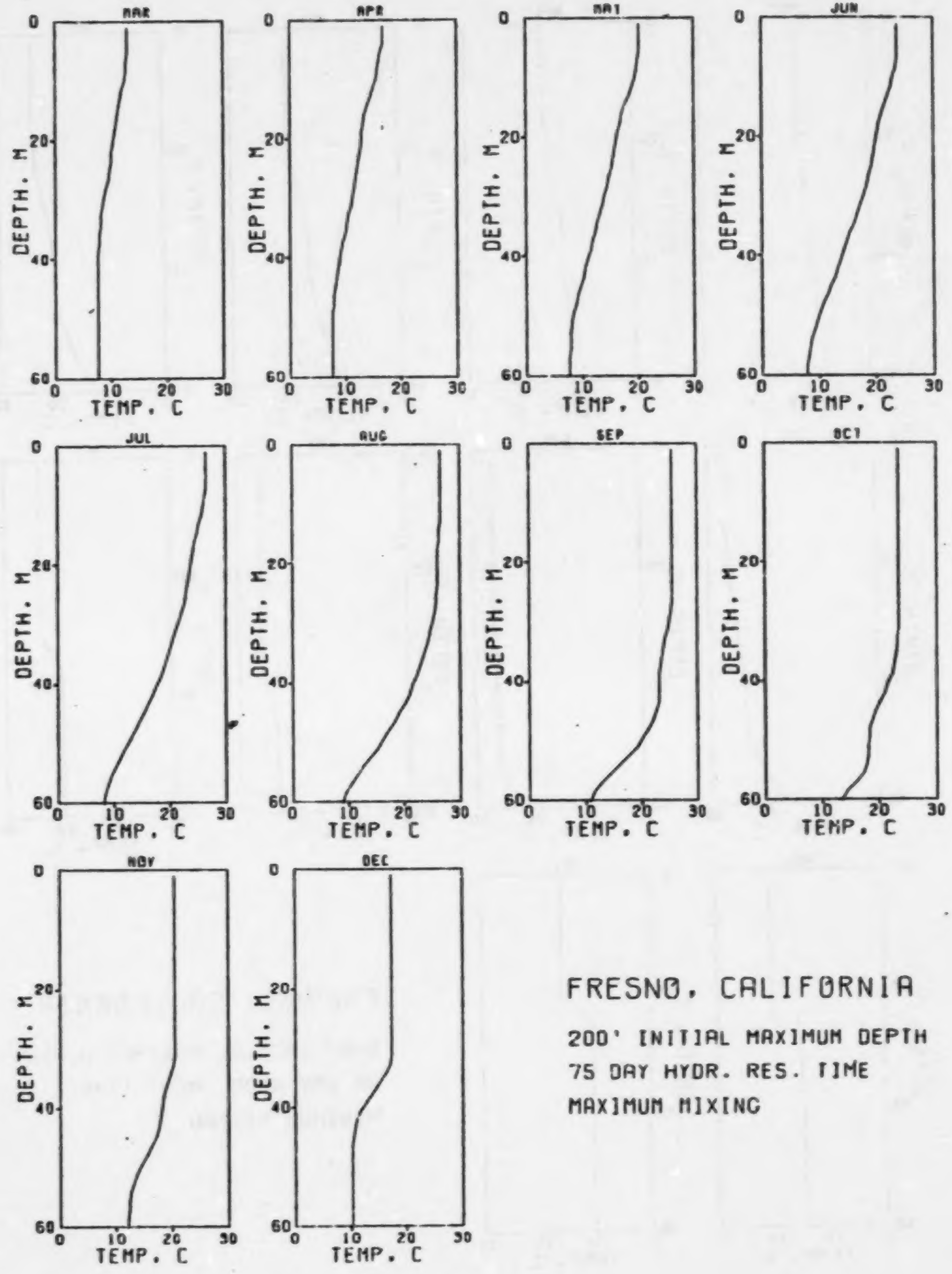






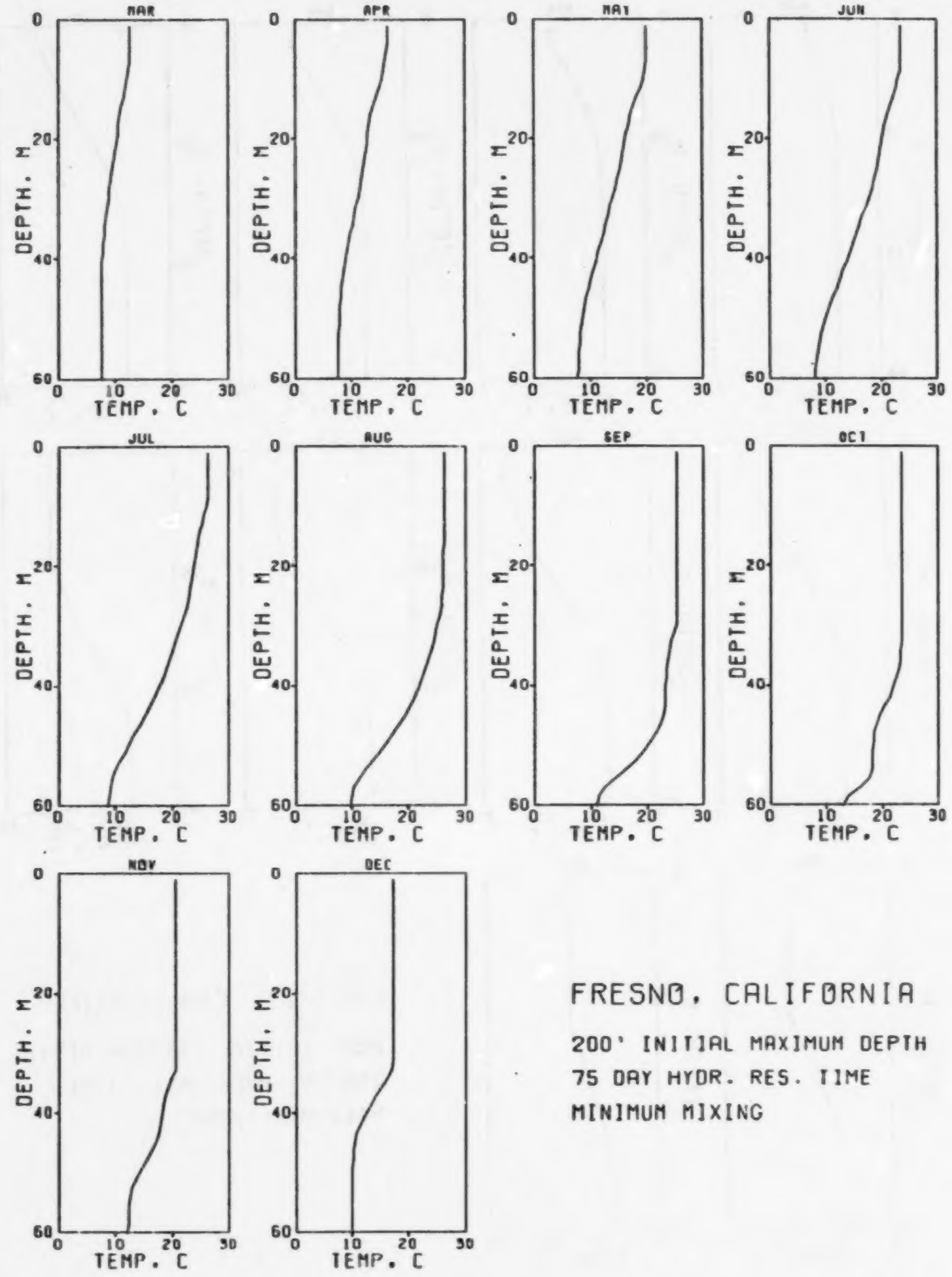
FRESNO, CALIFORNIA

200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



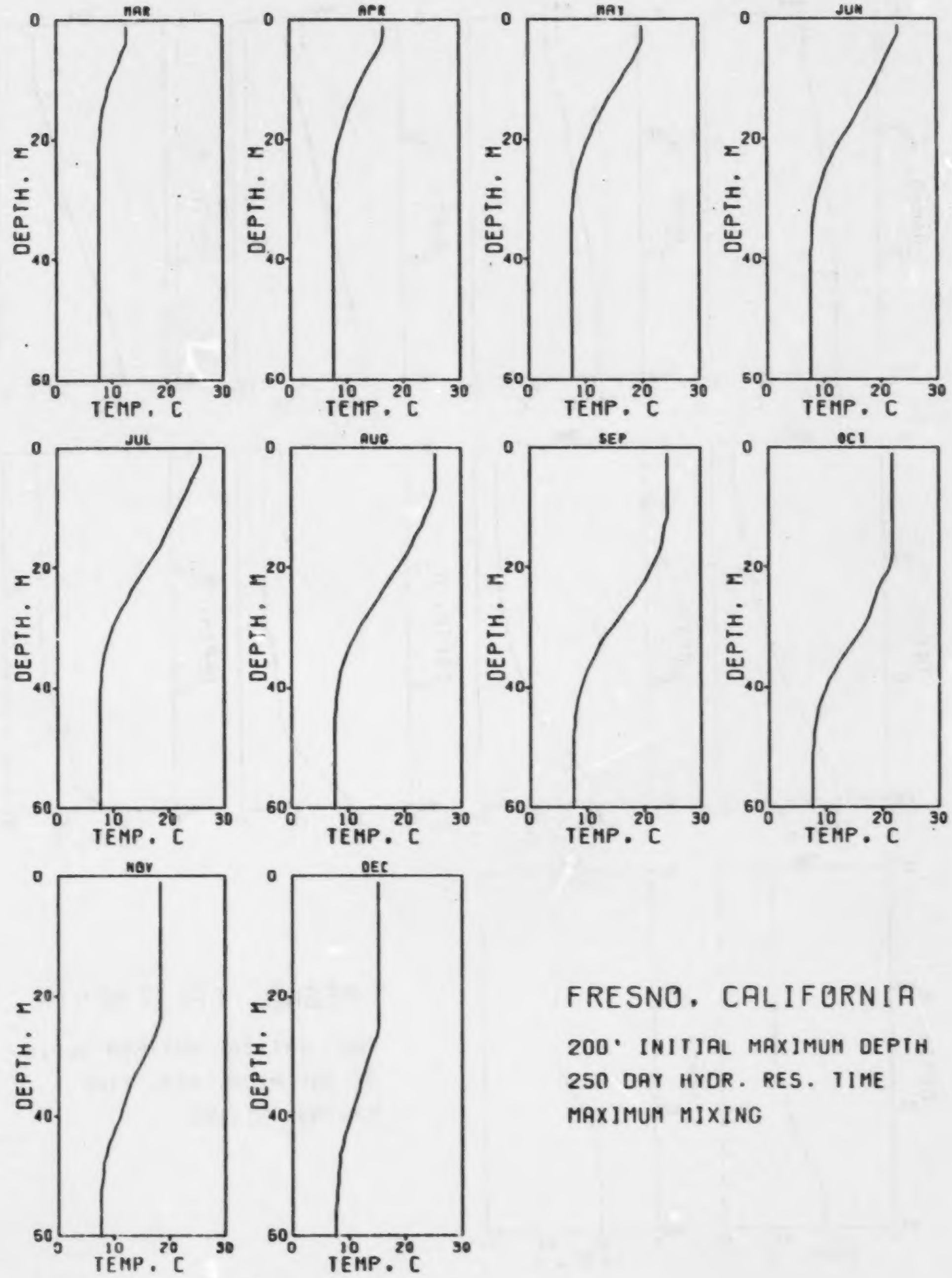
FRESNO, CALIFORNIA

200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



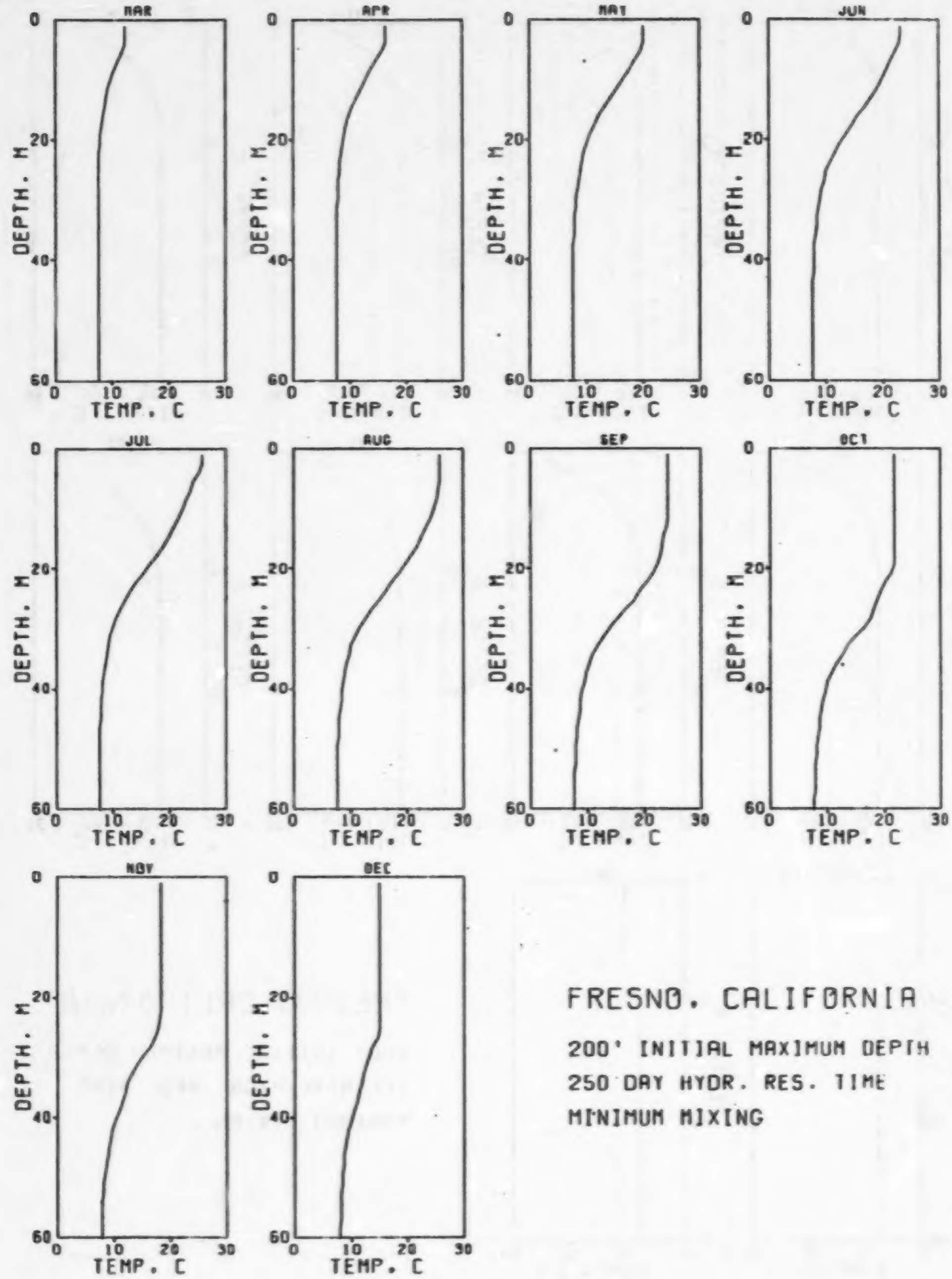
FRESNO, CALIFORNIA

200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



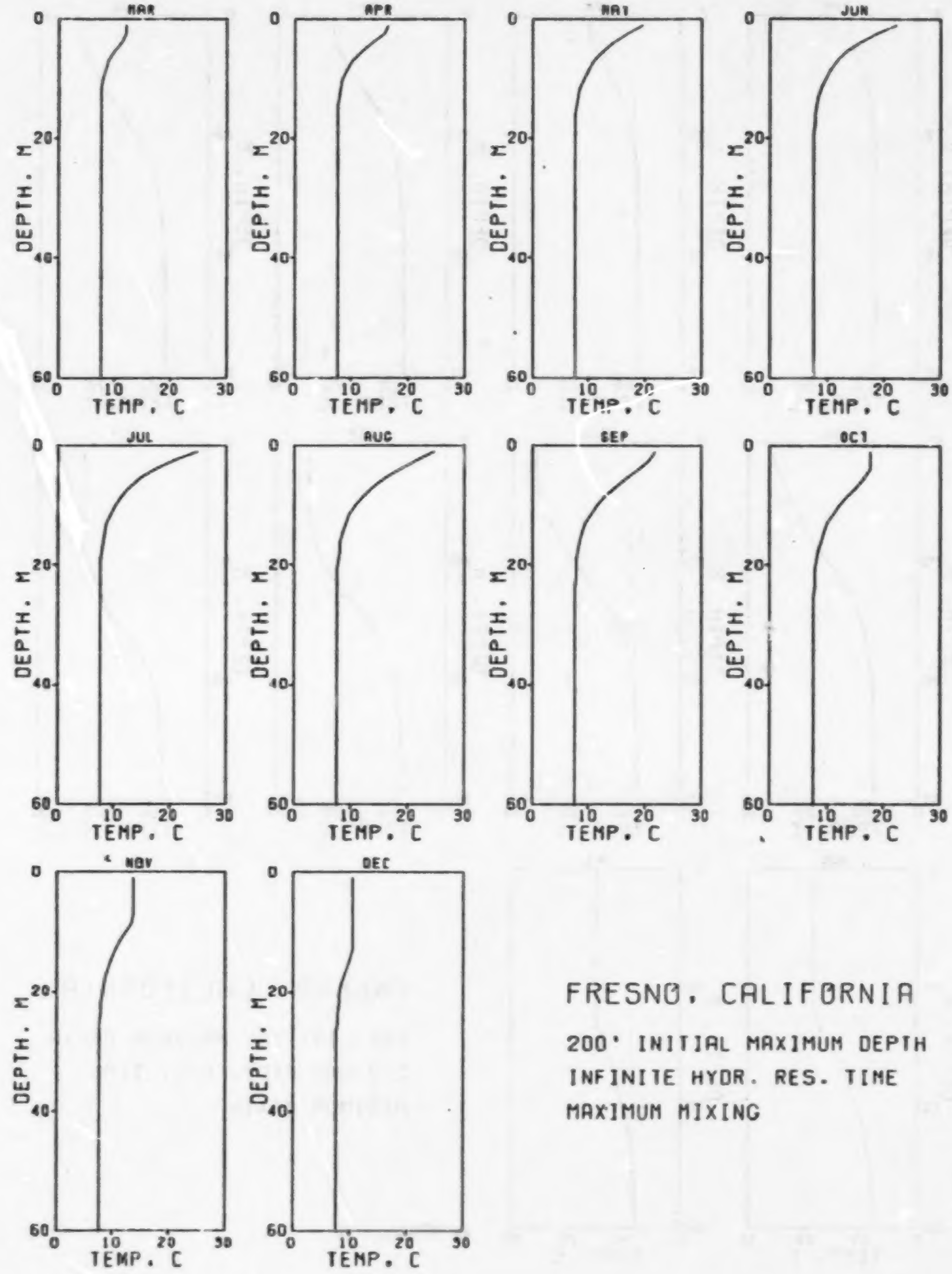
FRESNO, CALIFORNIA

200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



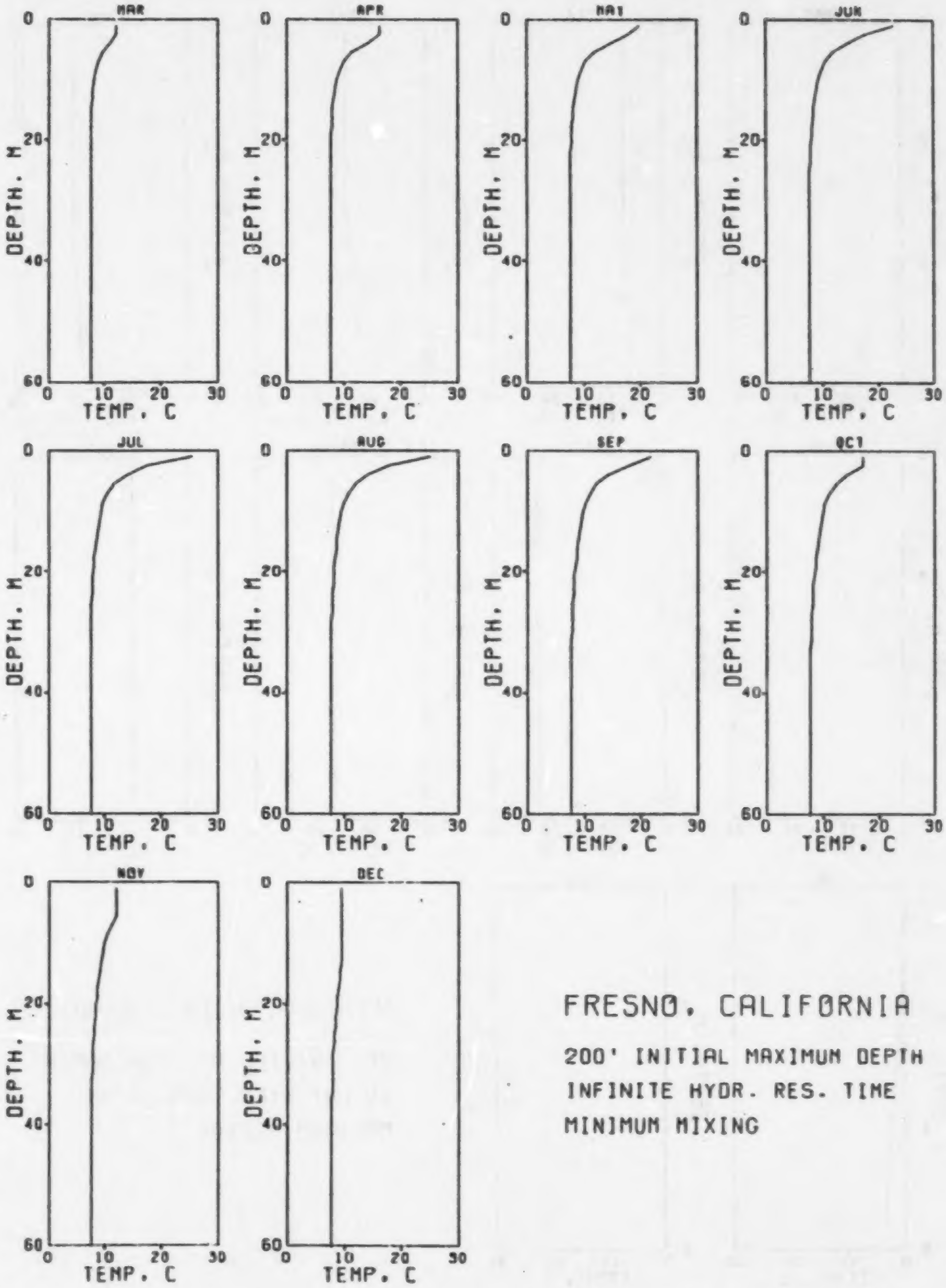
FRESNO, CALIFORNIA

200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



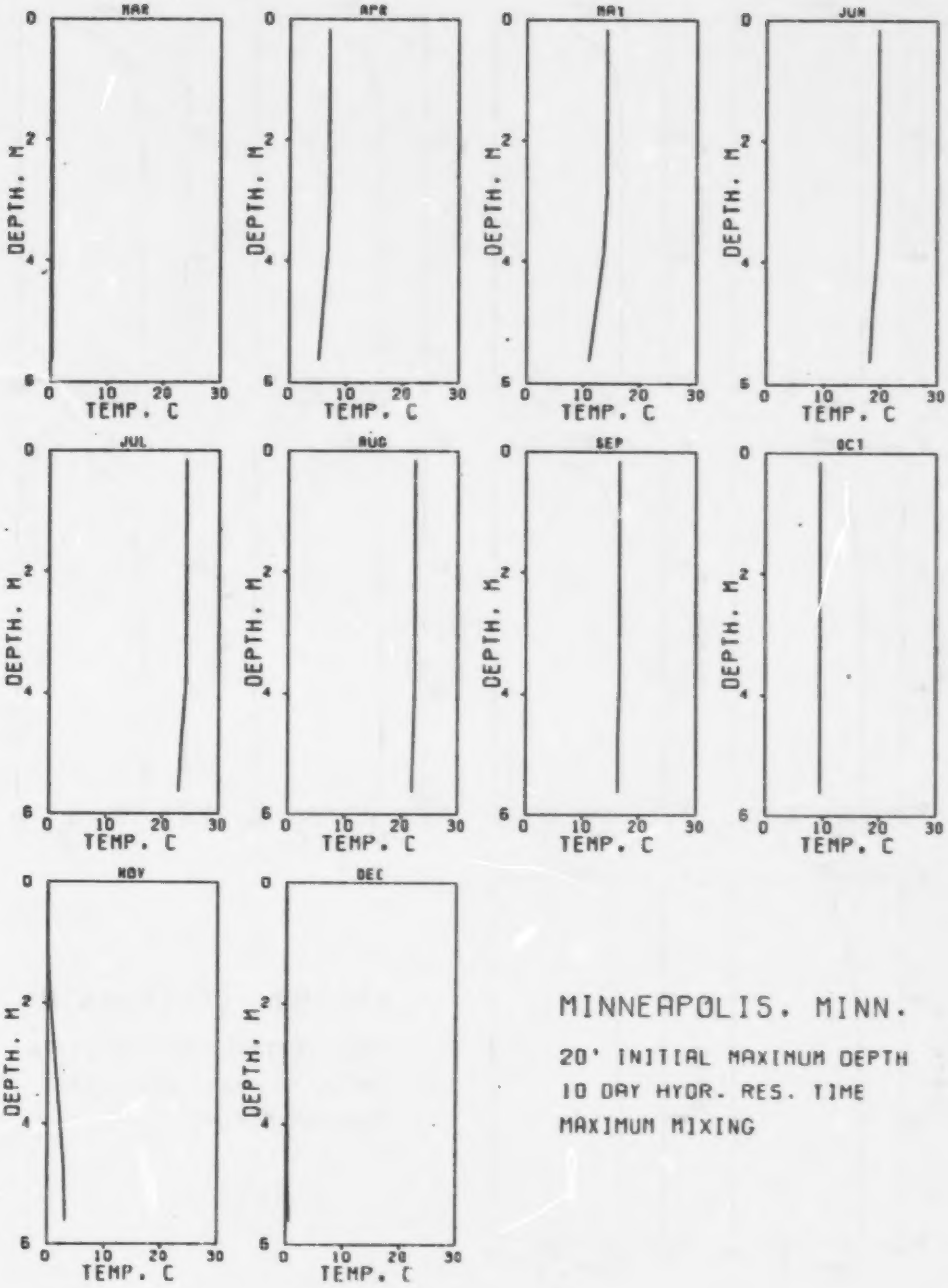
FRESNO, CALIFORNIA

200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



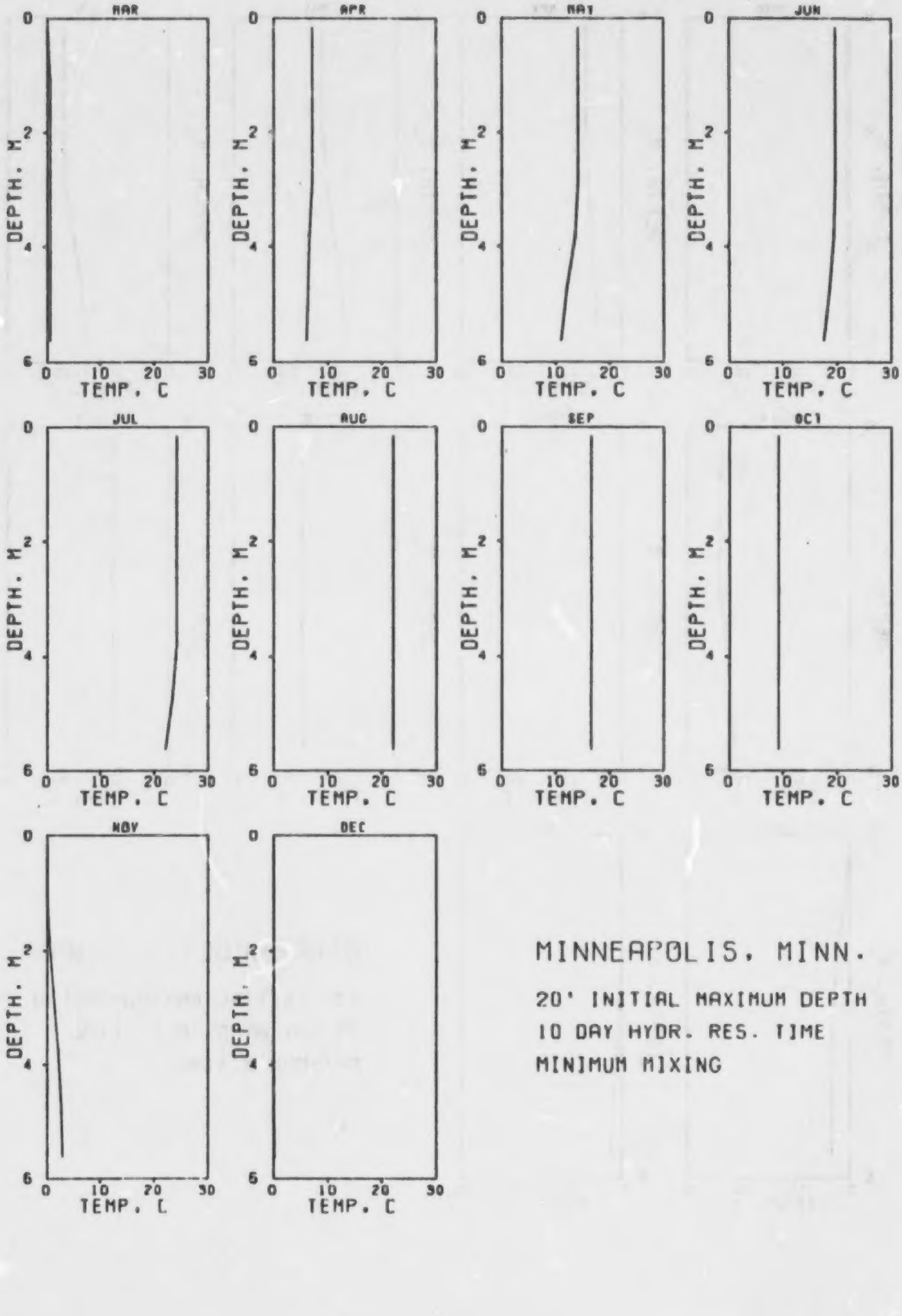
FRESNO, CALIFORNIA

200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



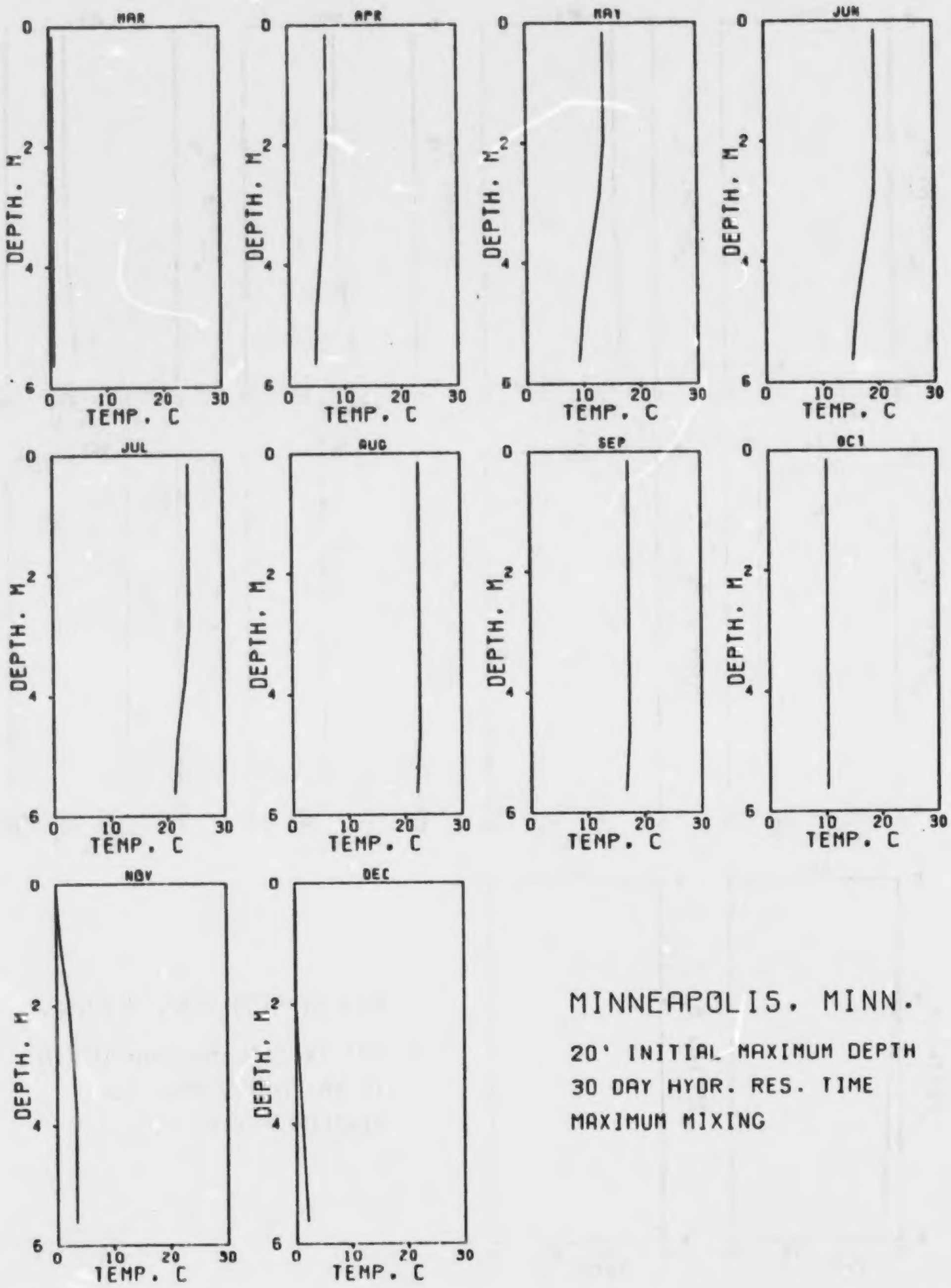
MINNEAPOLIS. MINN.

20' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



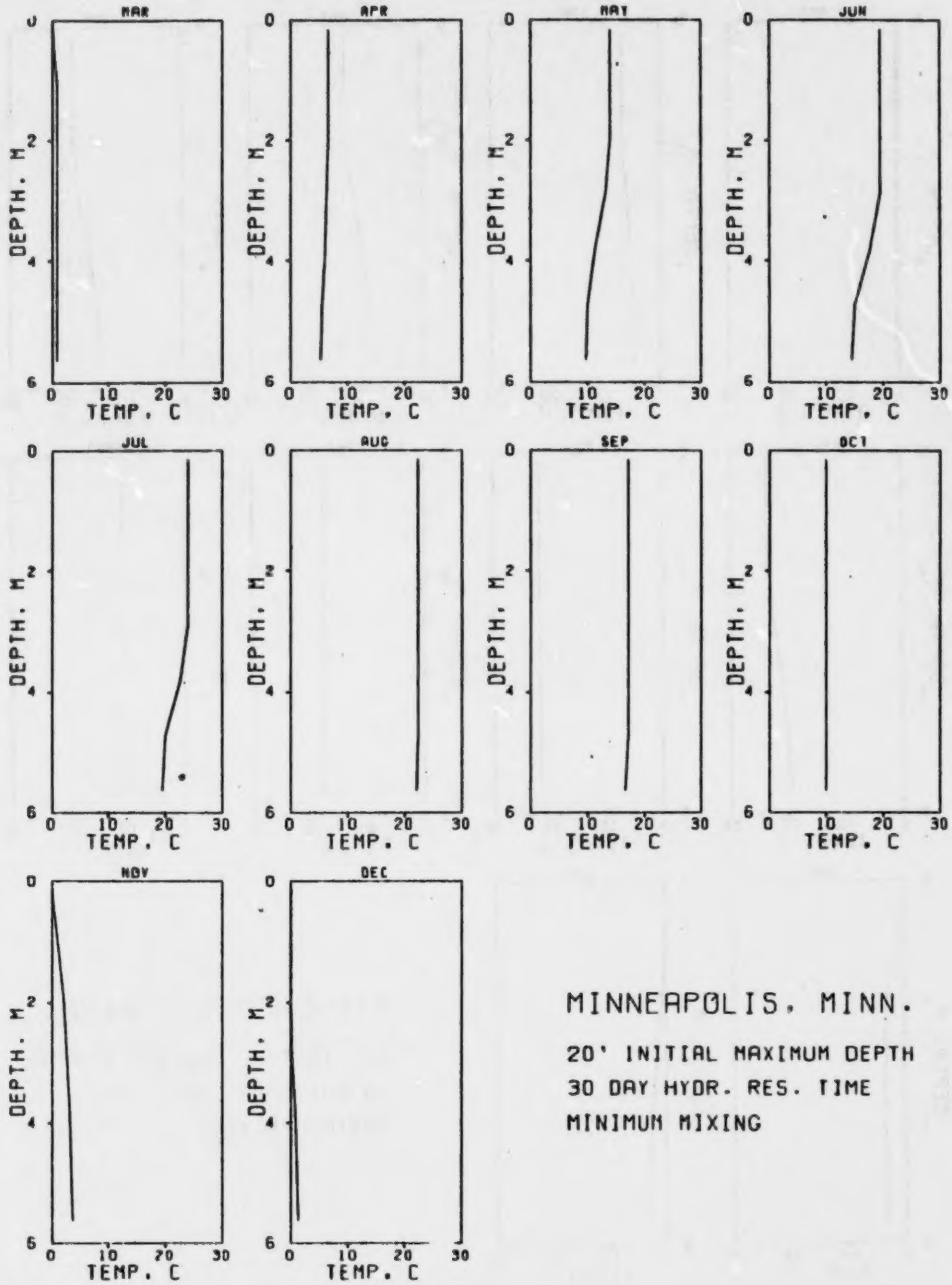
MINNEAPOLIS, MINN.

20' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



MINNEAPOLIS. MINN.

20' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



MINNEAPOLIS, MINN.

20' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING

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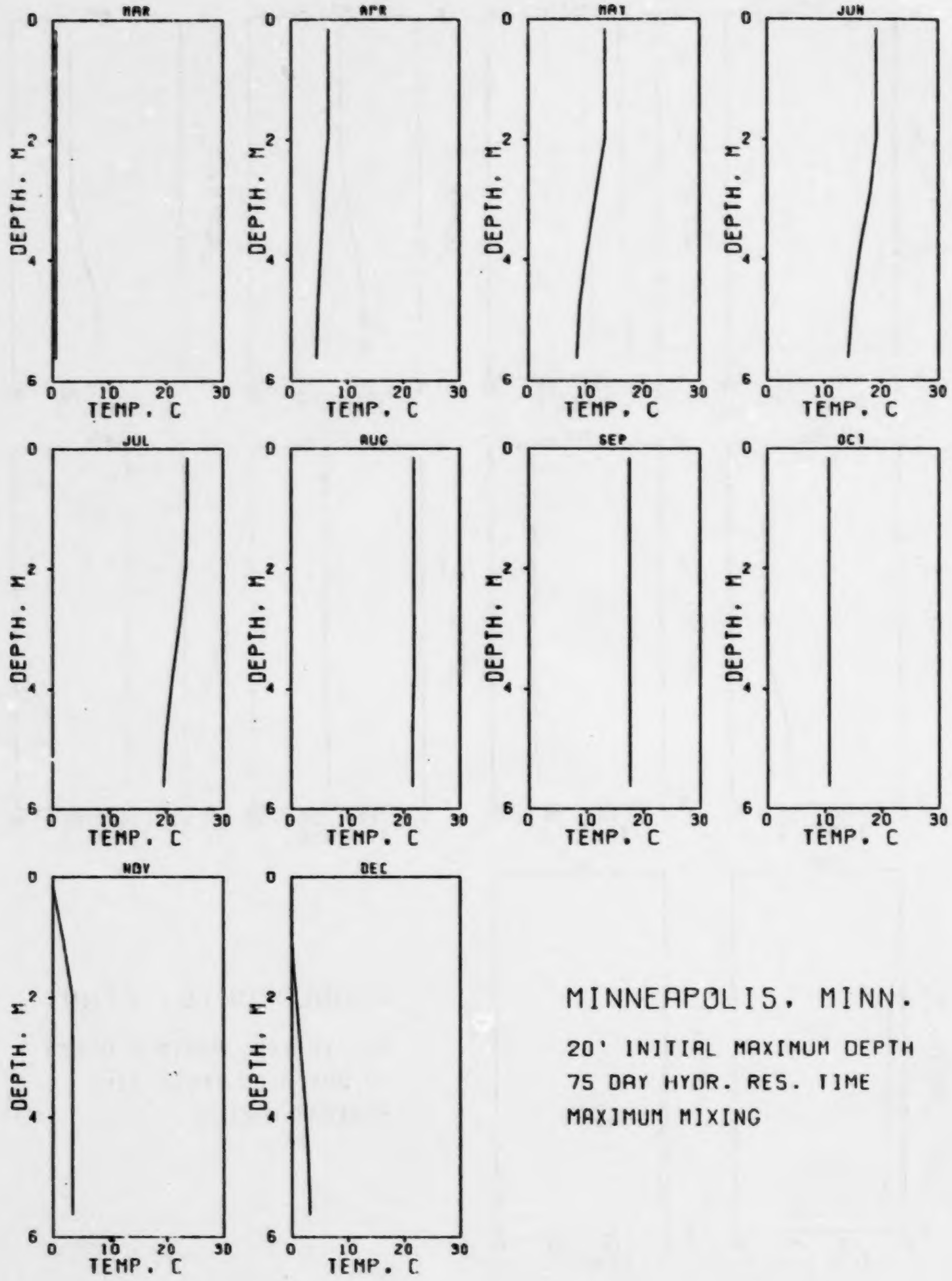
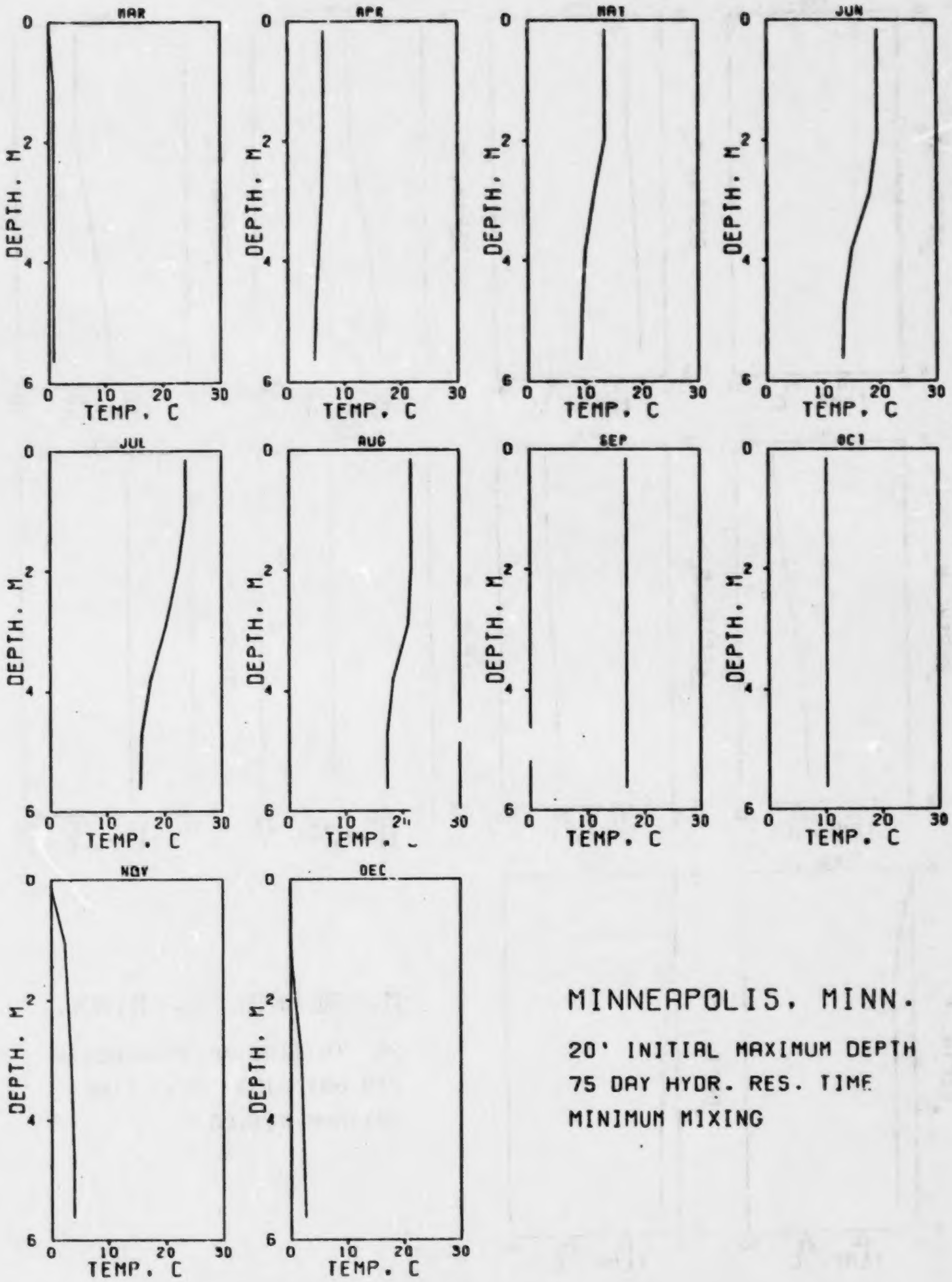


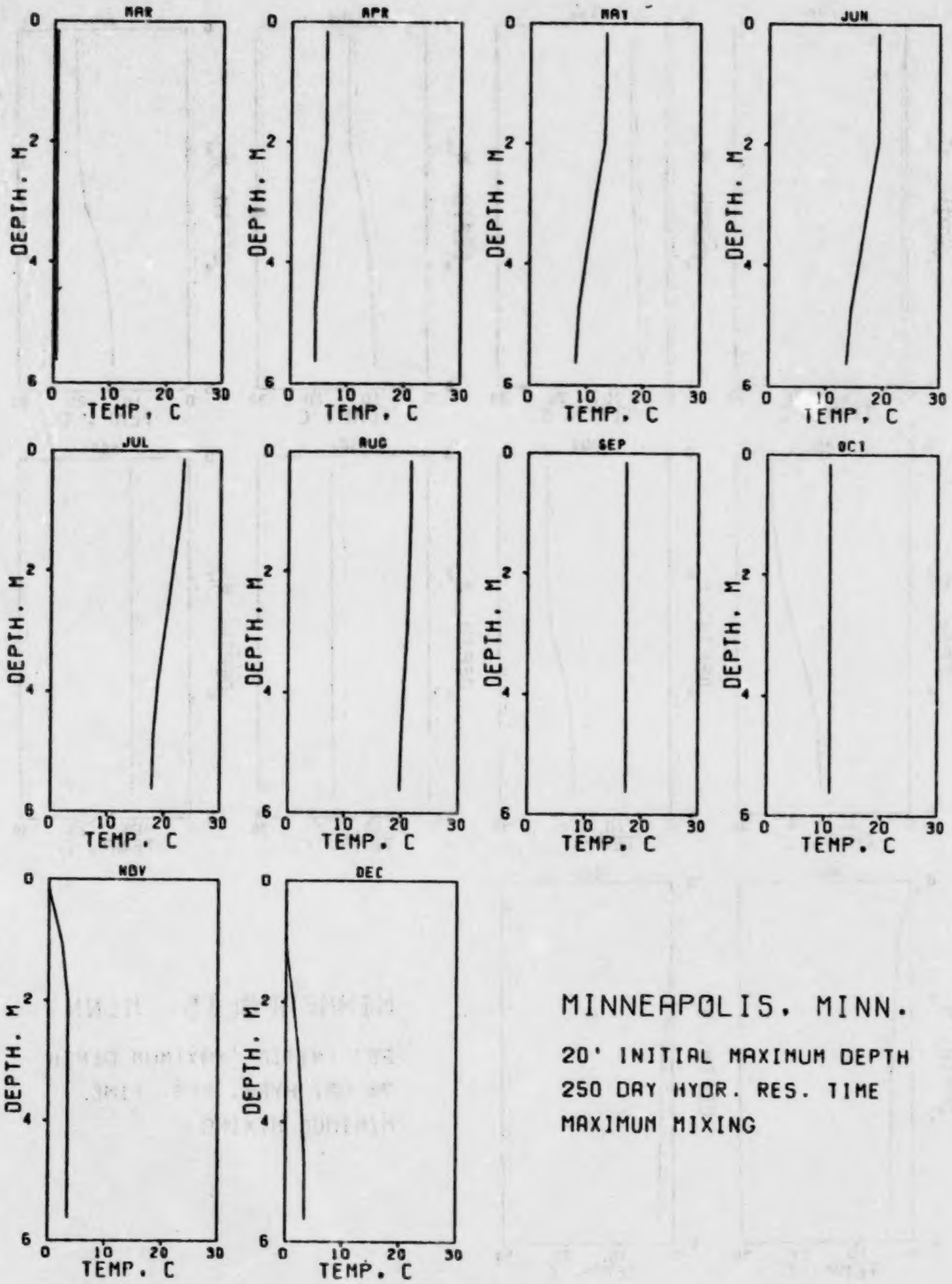
MINNEAPOLIS, MINN.

20' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING

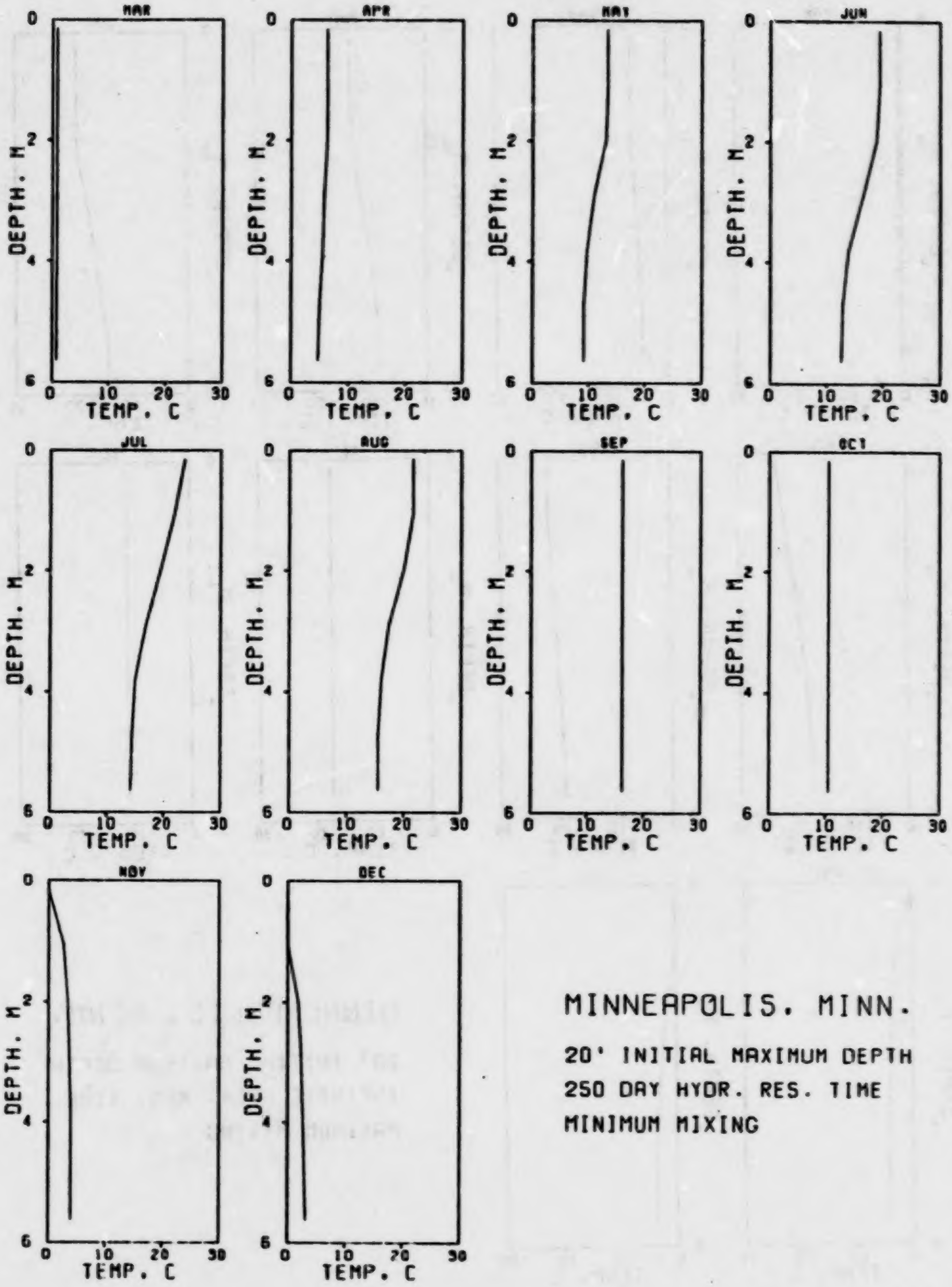


MINNEAPOLIS, MINN.

20' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING

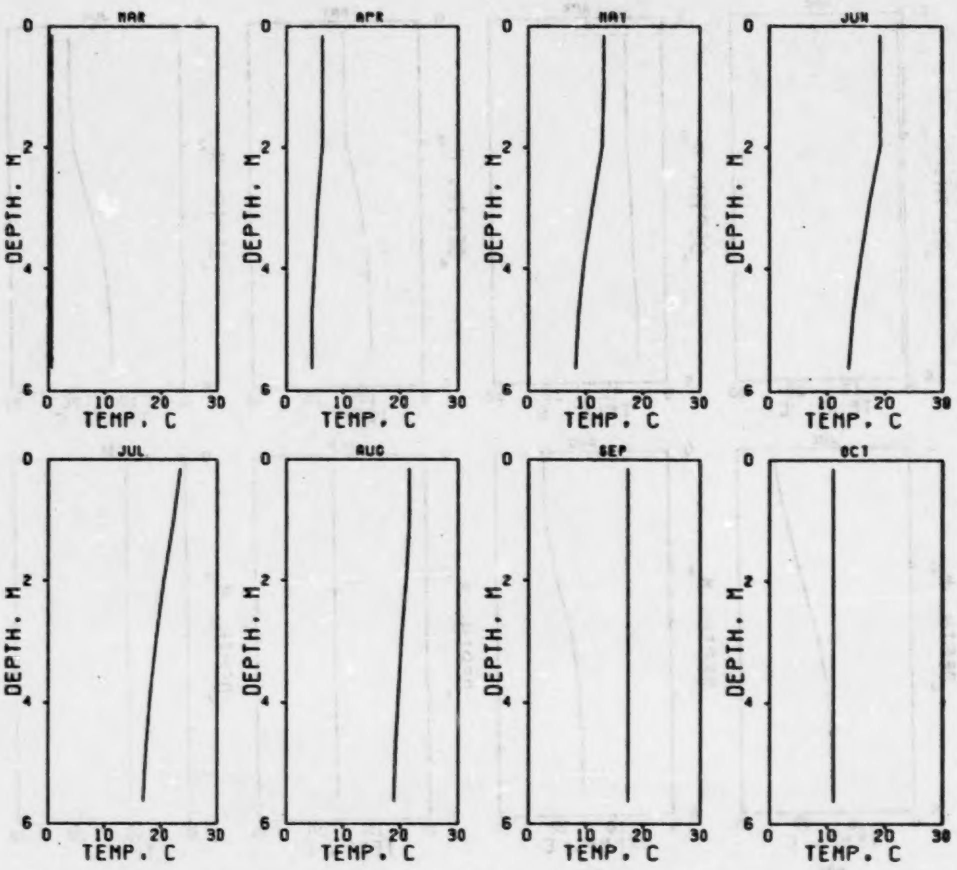


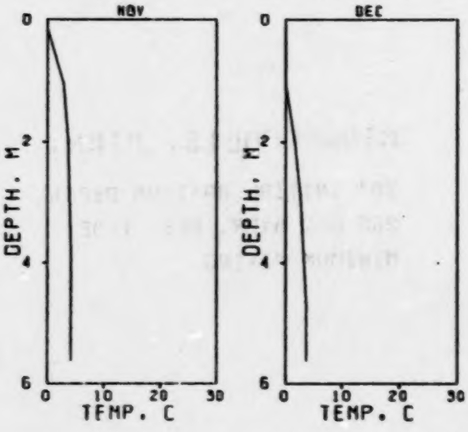
MINNEAPOLIS, MINN.
20' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



MINNEAPOLIS, MINN.

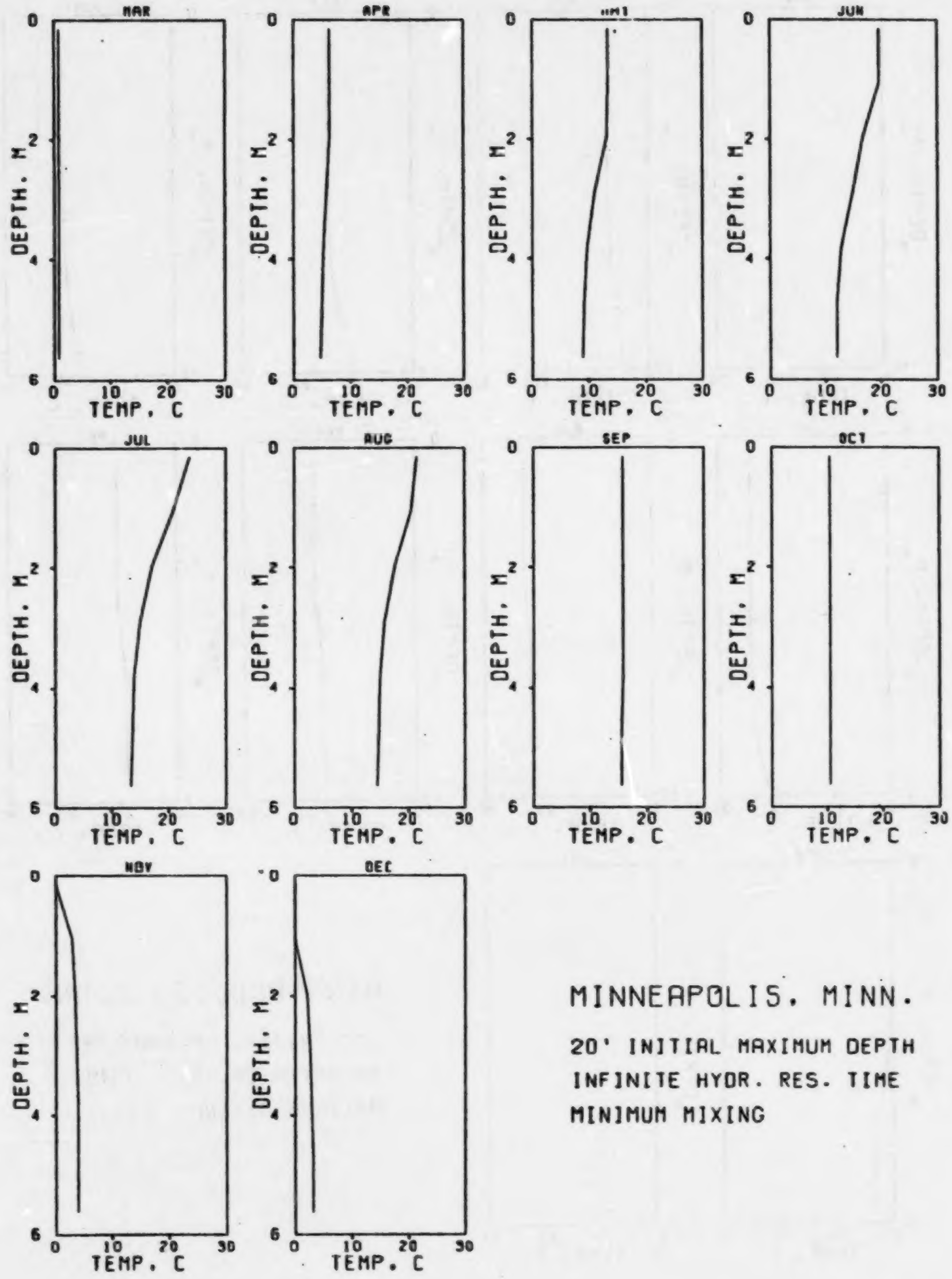
20' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING





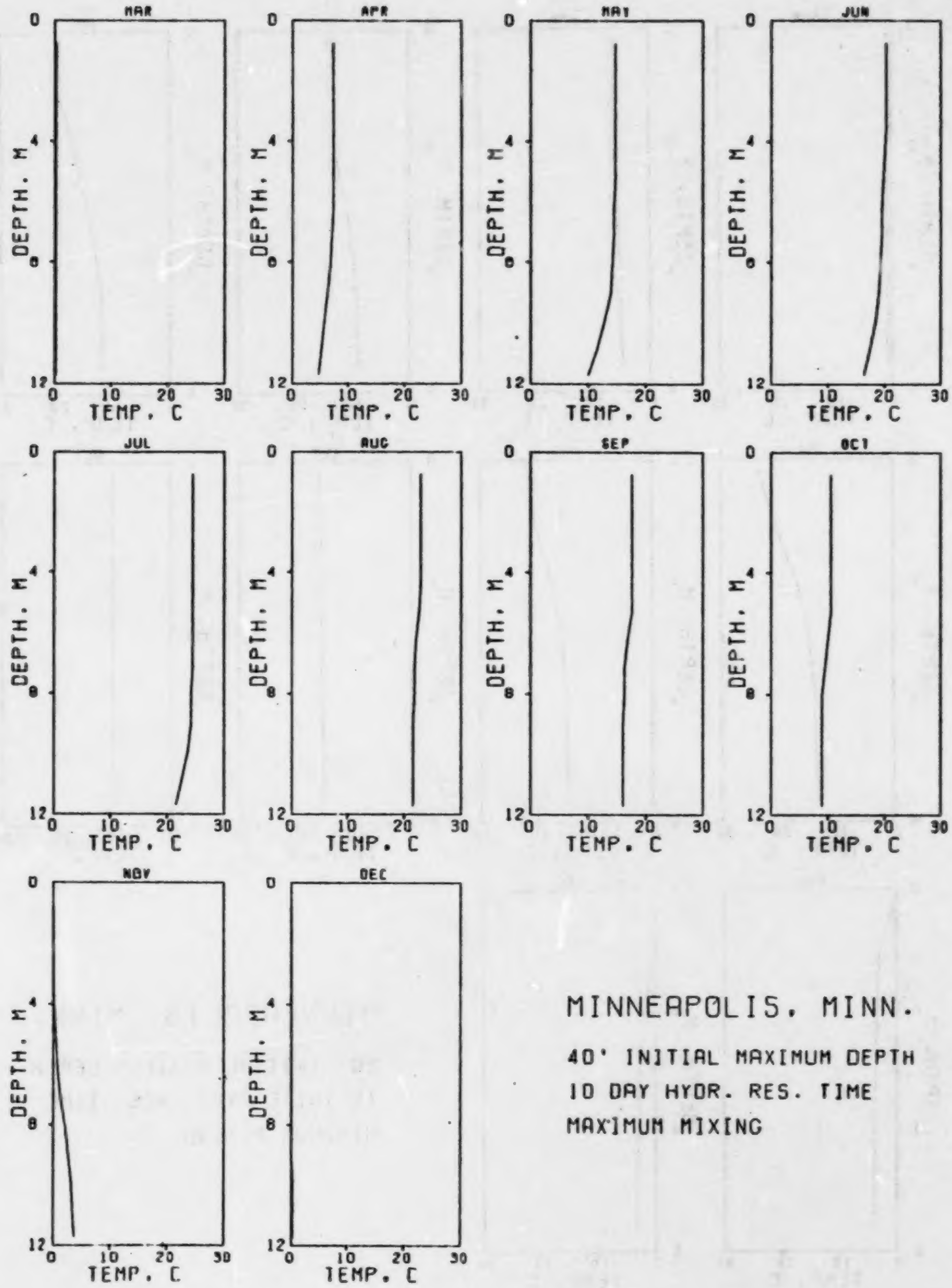
MINNEAPOLIS, MINN.

20' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



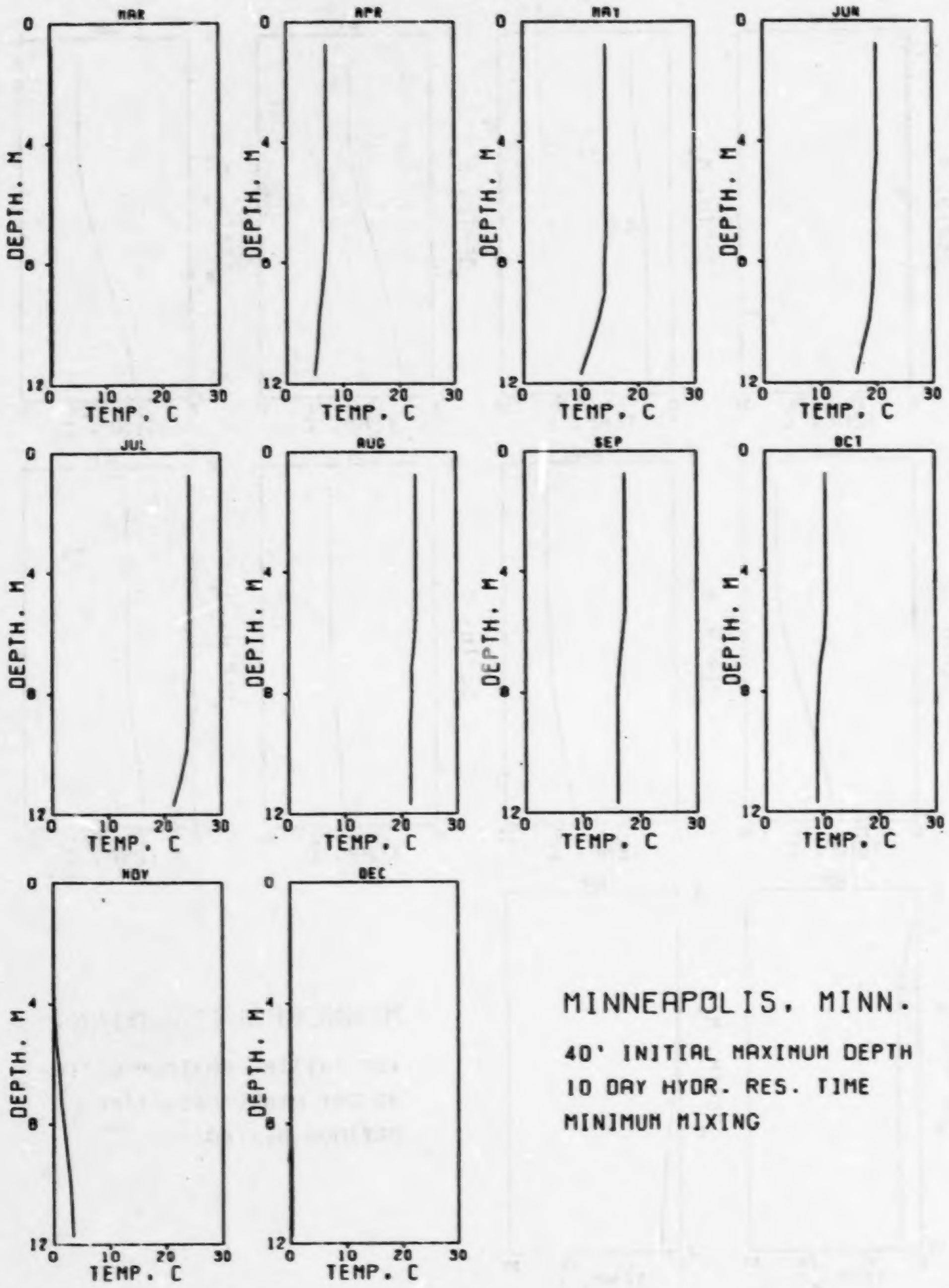
MINNEAPOLIS, MINN.

20' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



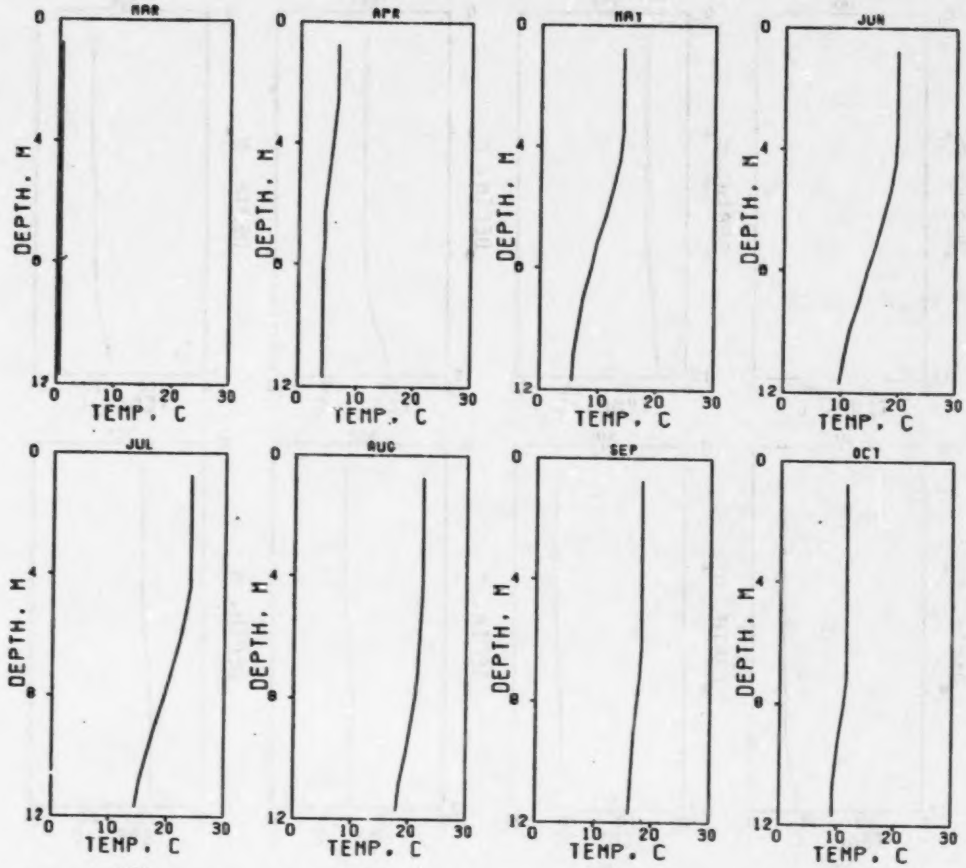
MINNEAPOLIS. MINN.

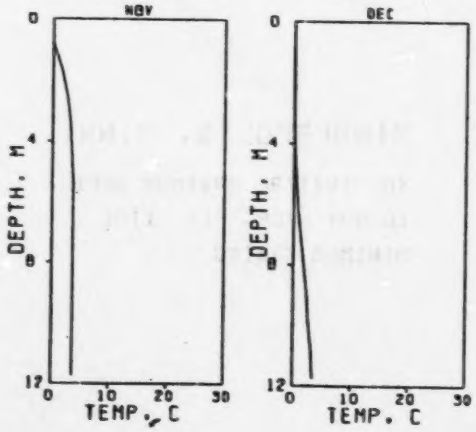
40' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



MINNEAPOLIS, MINN.

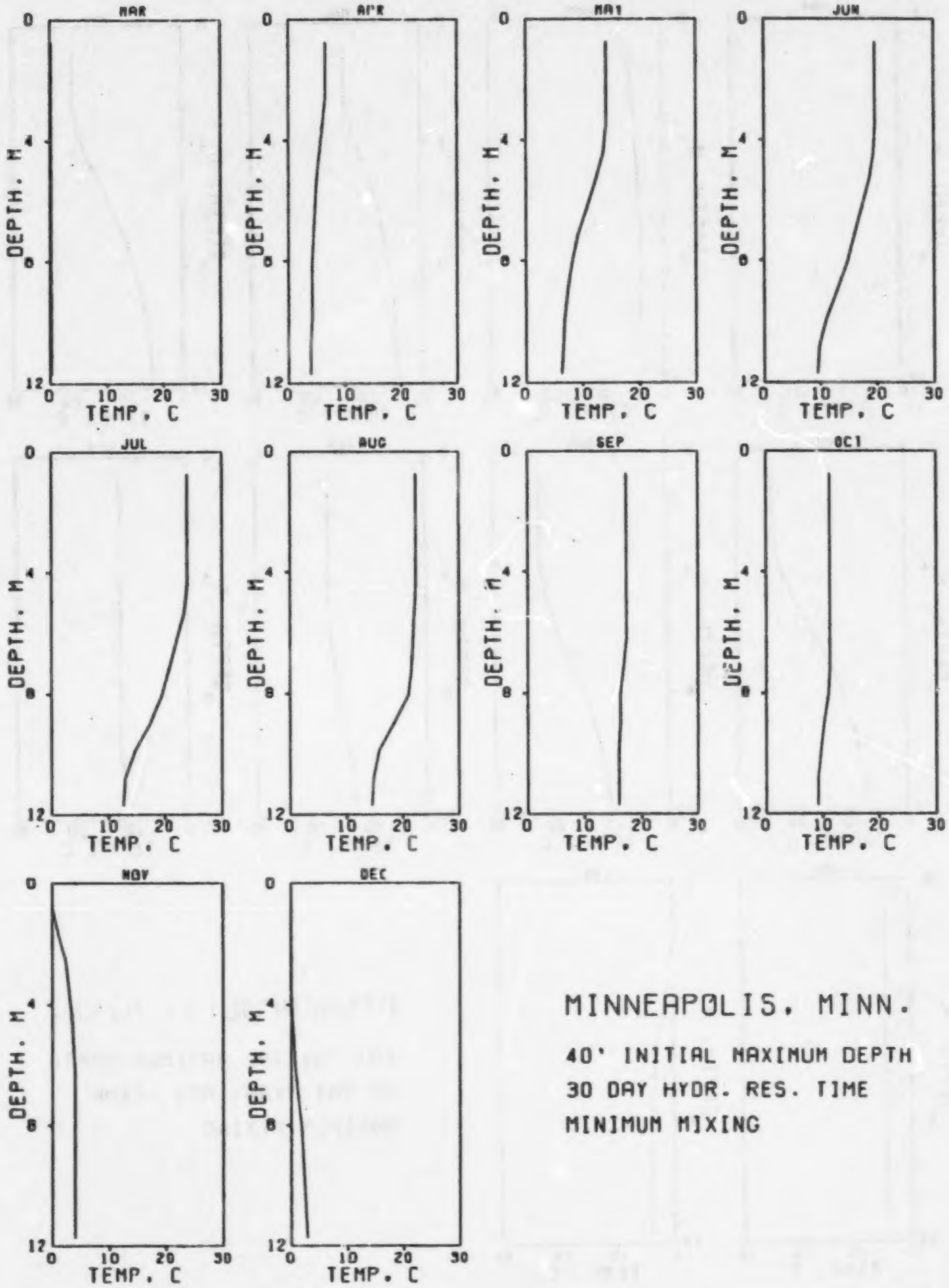
40' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING





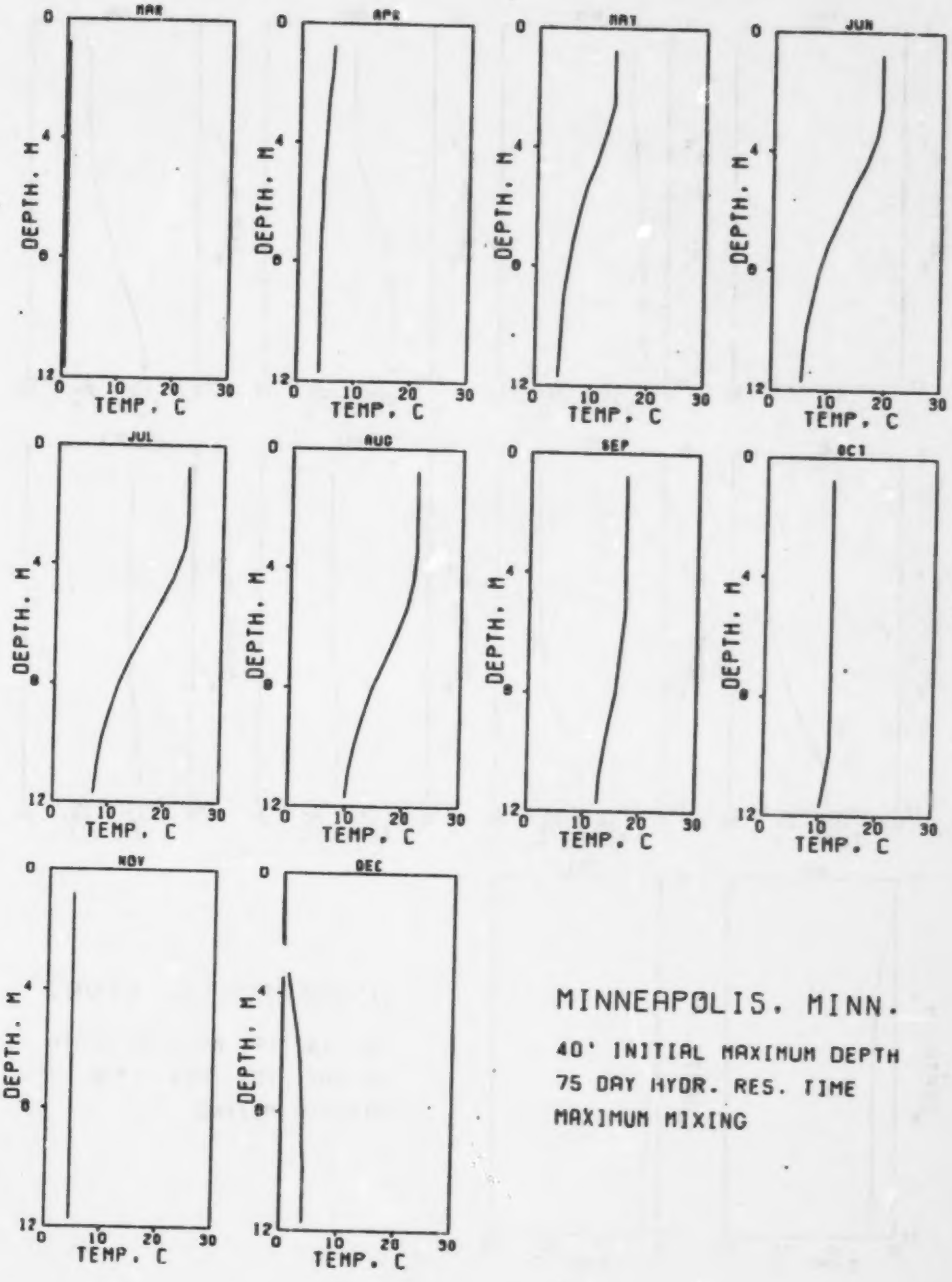
MINNEAPOLIS, MINN.

40' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



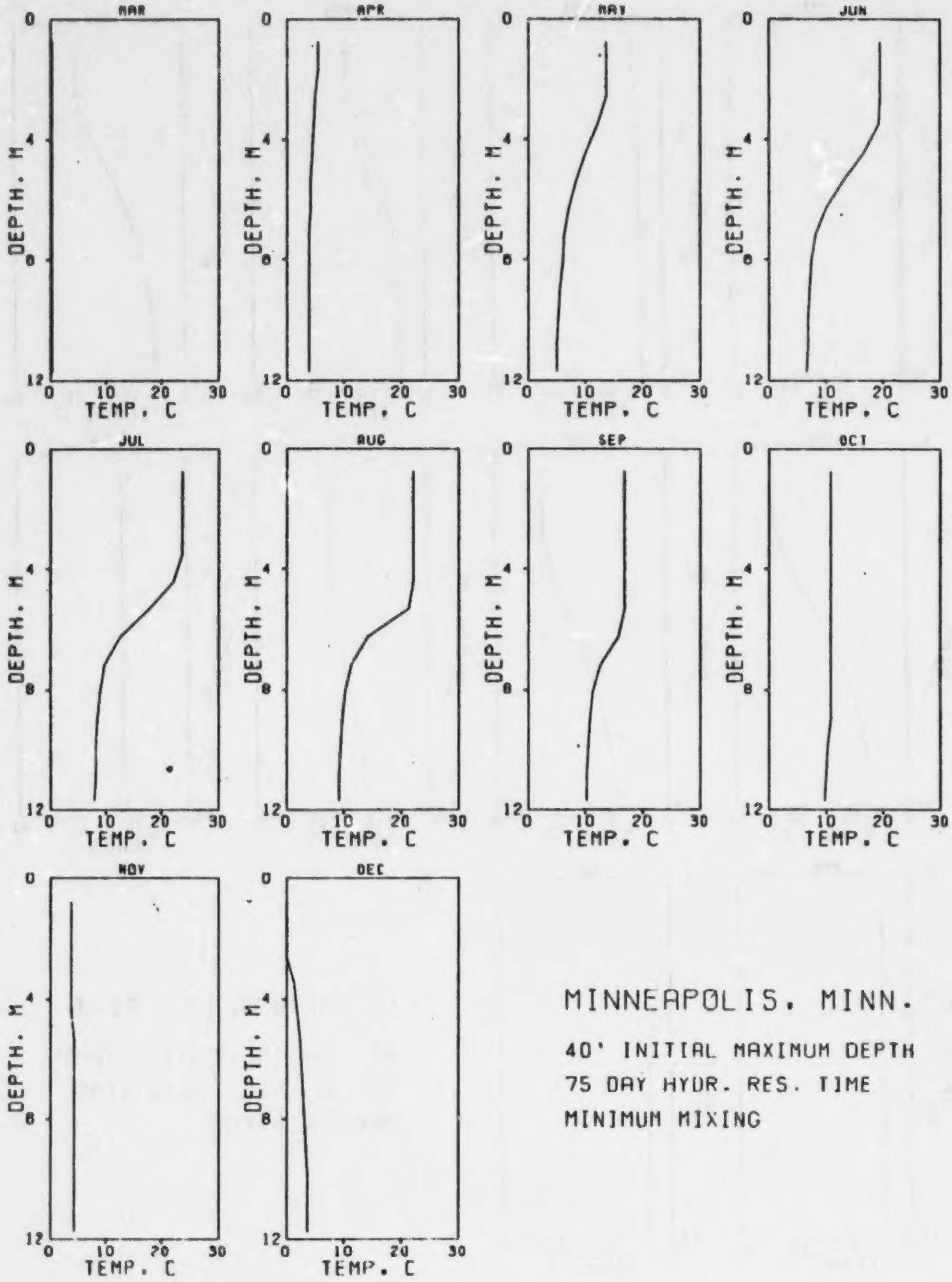
MINNEAPOLIS, MINN.

40' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



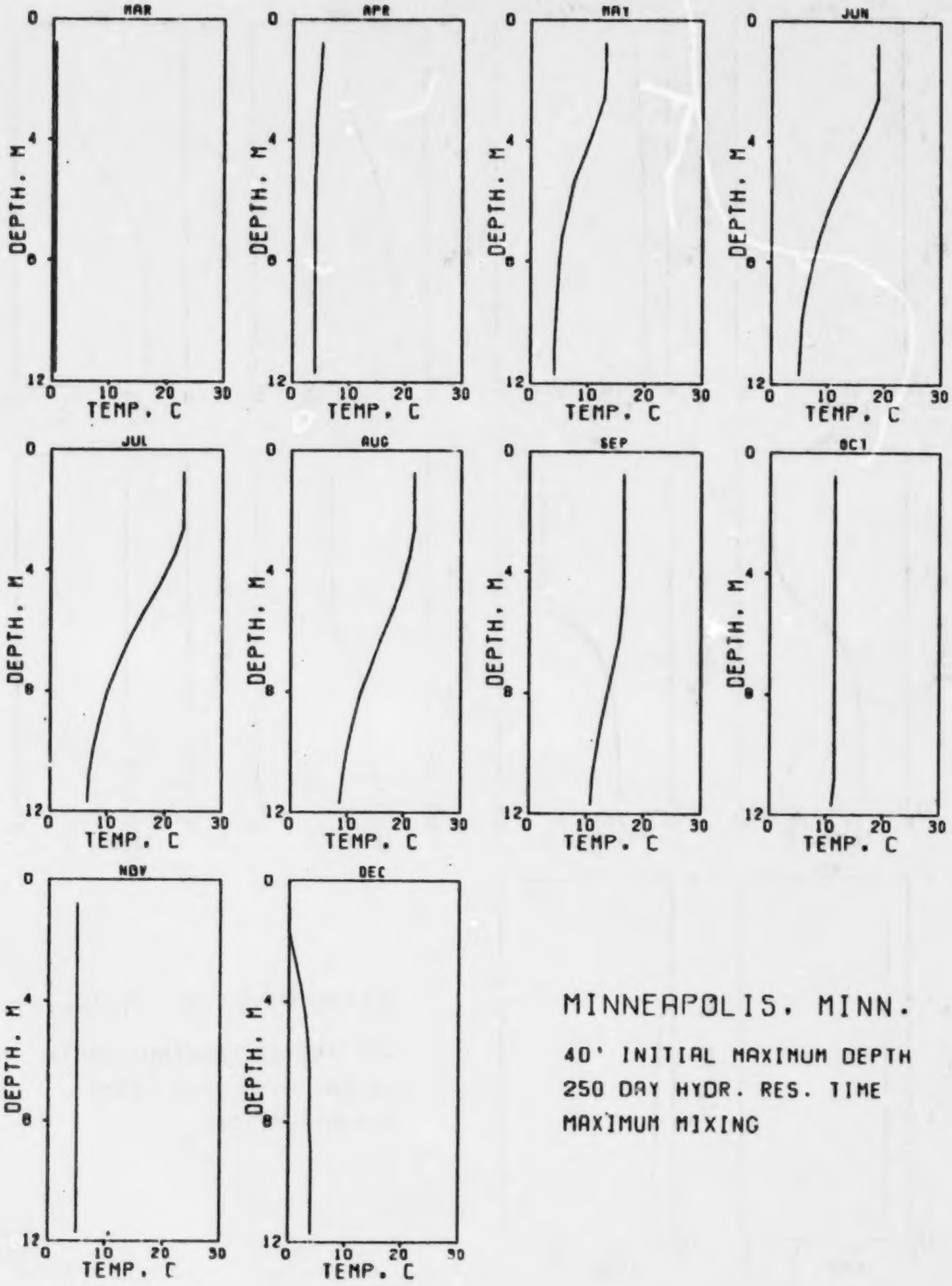
MINNEAPOLIS, MINN.

40' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



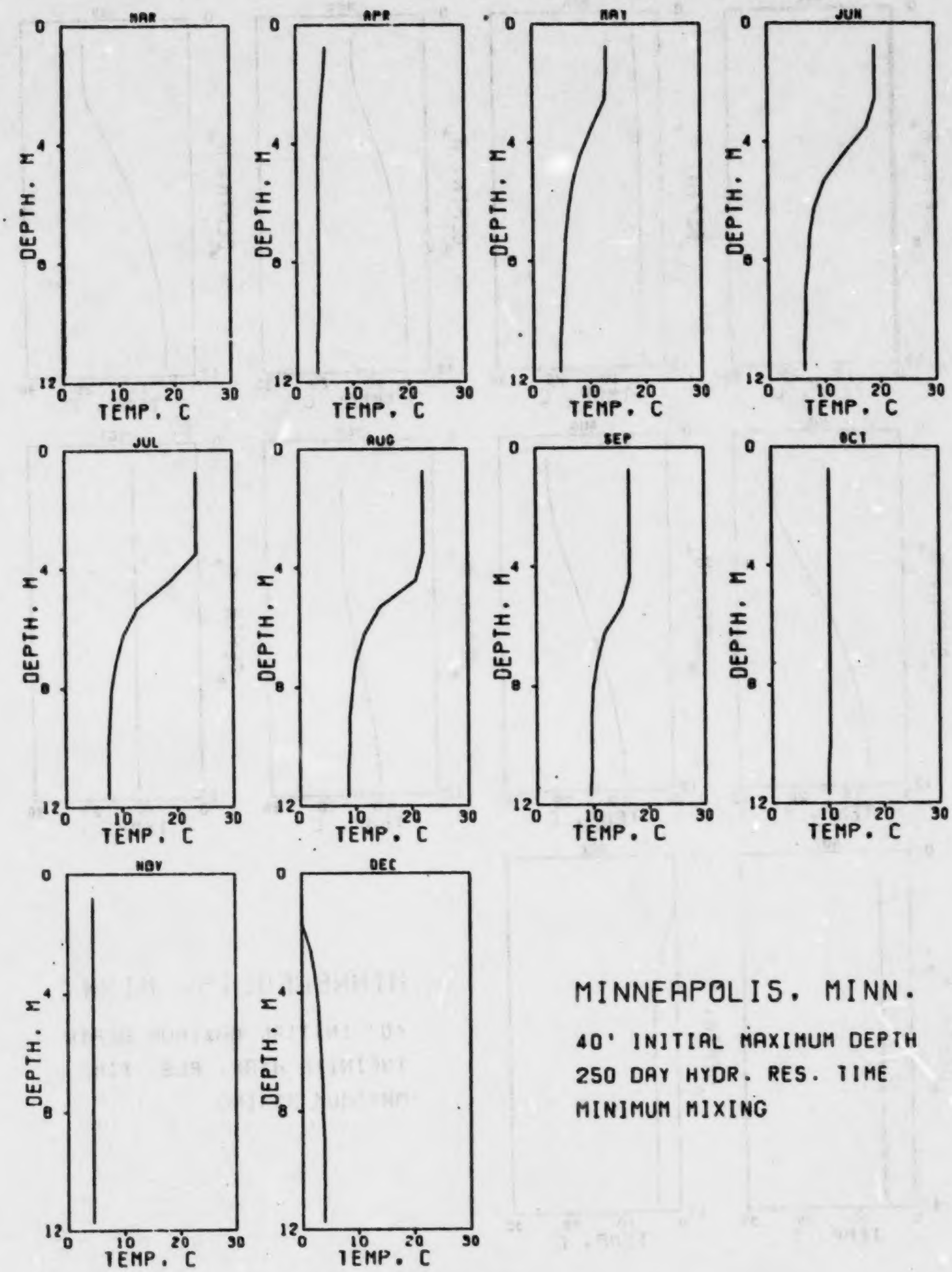
MINNEAPOLIS, MINN.

40' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



MINNEAPOLIS, MINN.

40' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING

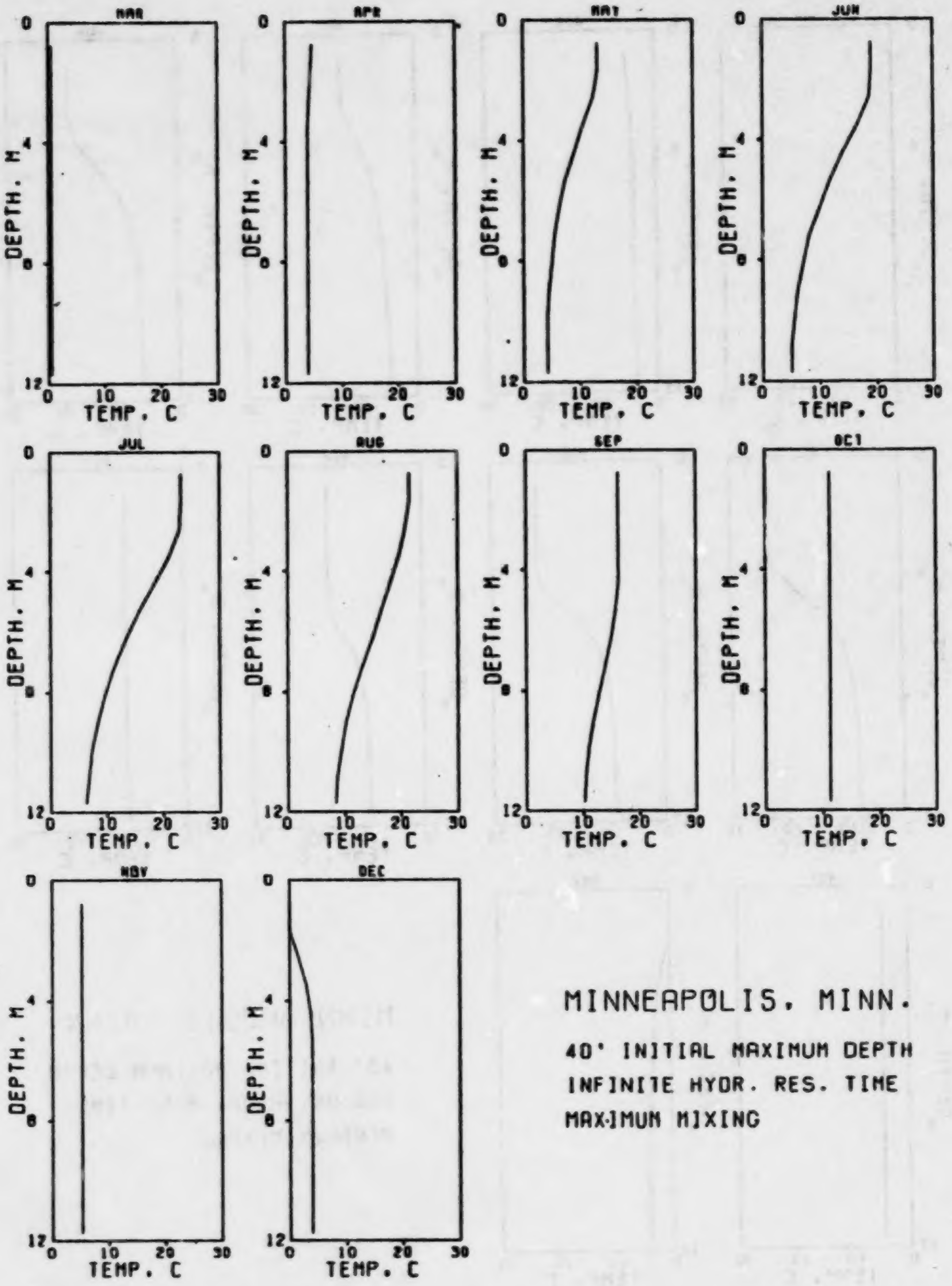


MINNEAPOLIS, MINN.

40' INITIAL MAXIMUM DEPTH

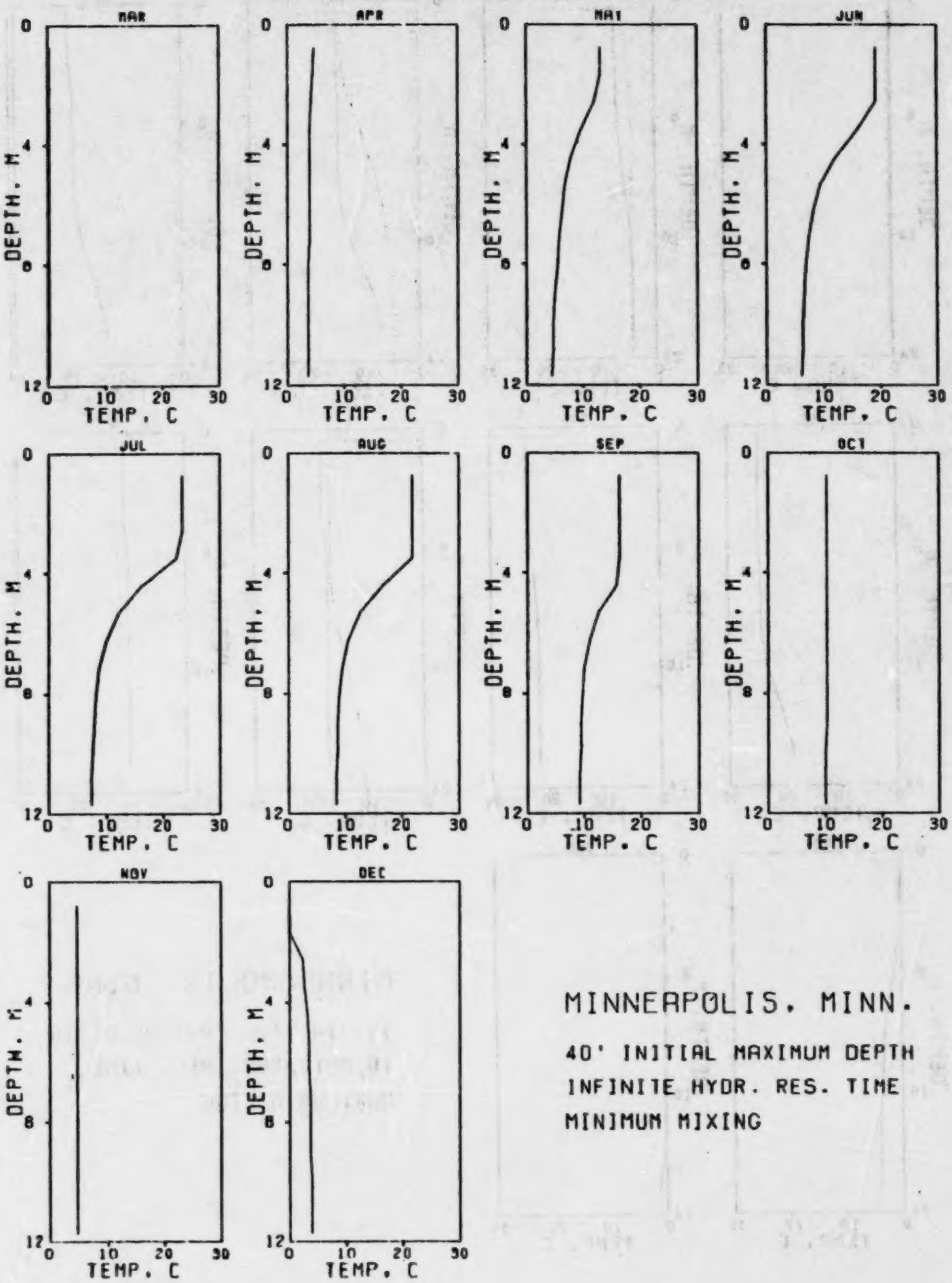
250 DAY HYDR. RES. TIME

MINIMUM MIXING



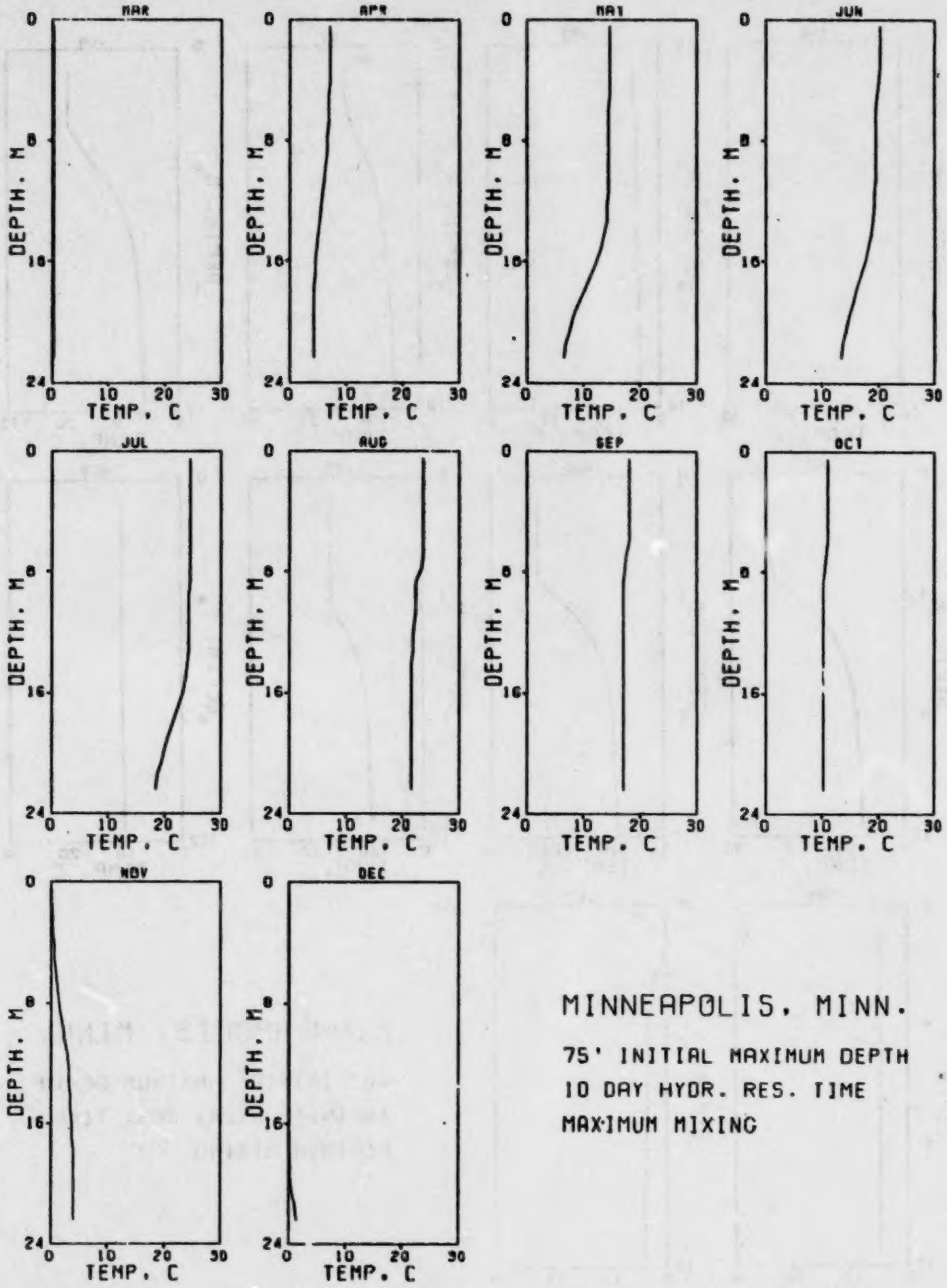
MINNEAPOLIS, MINN.

40' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



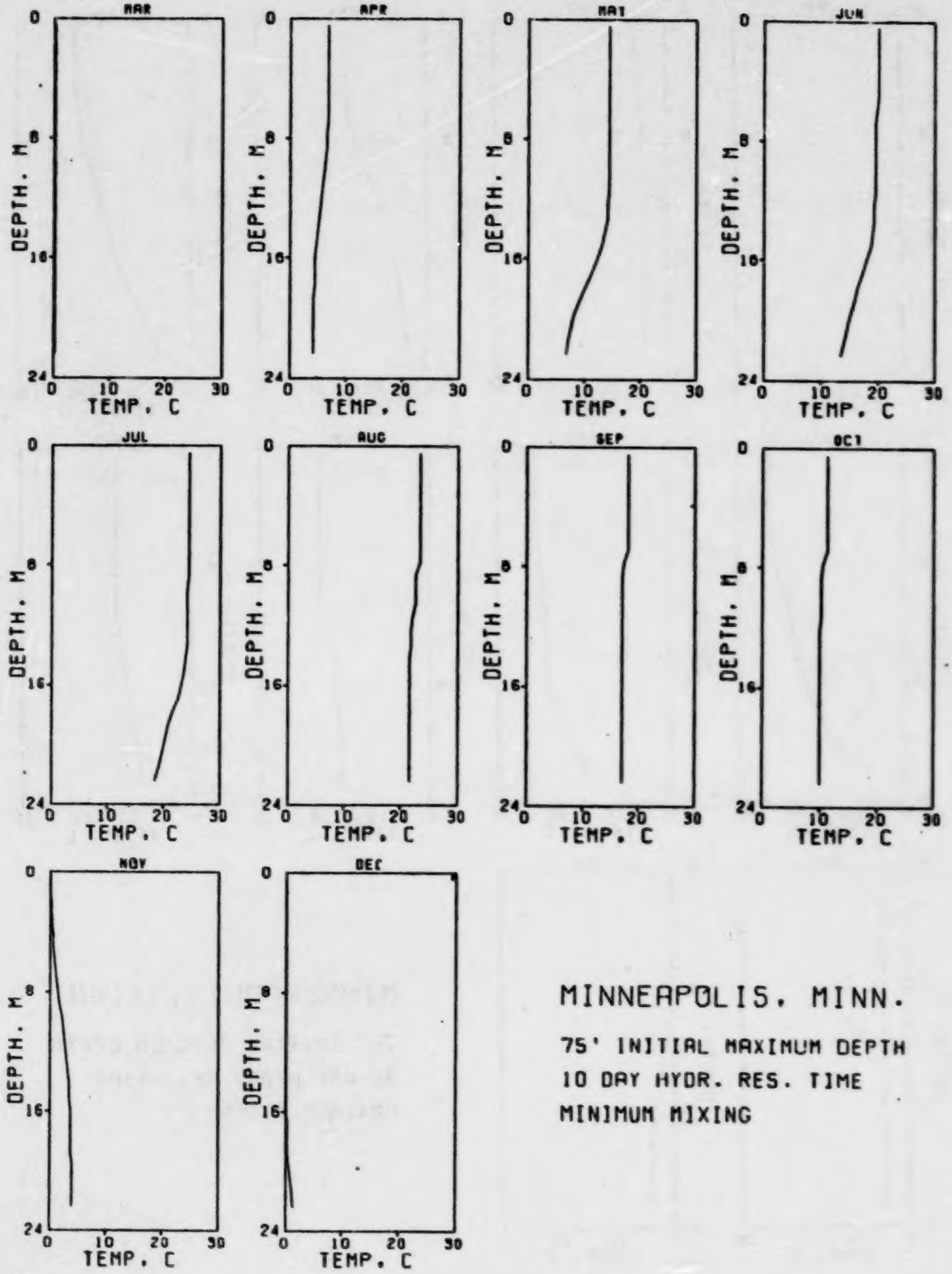
MINNEAPOLIS, MINN.

40' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



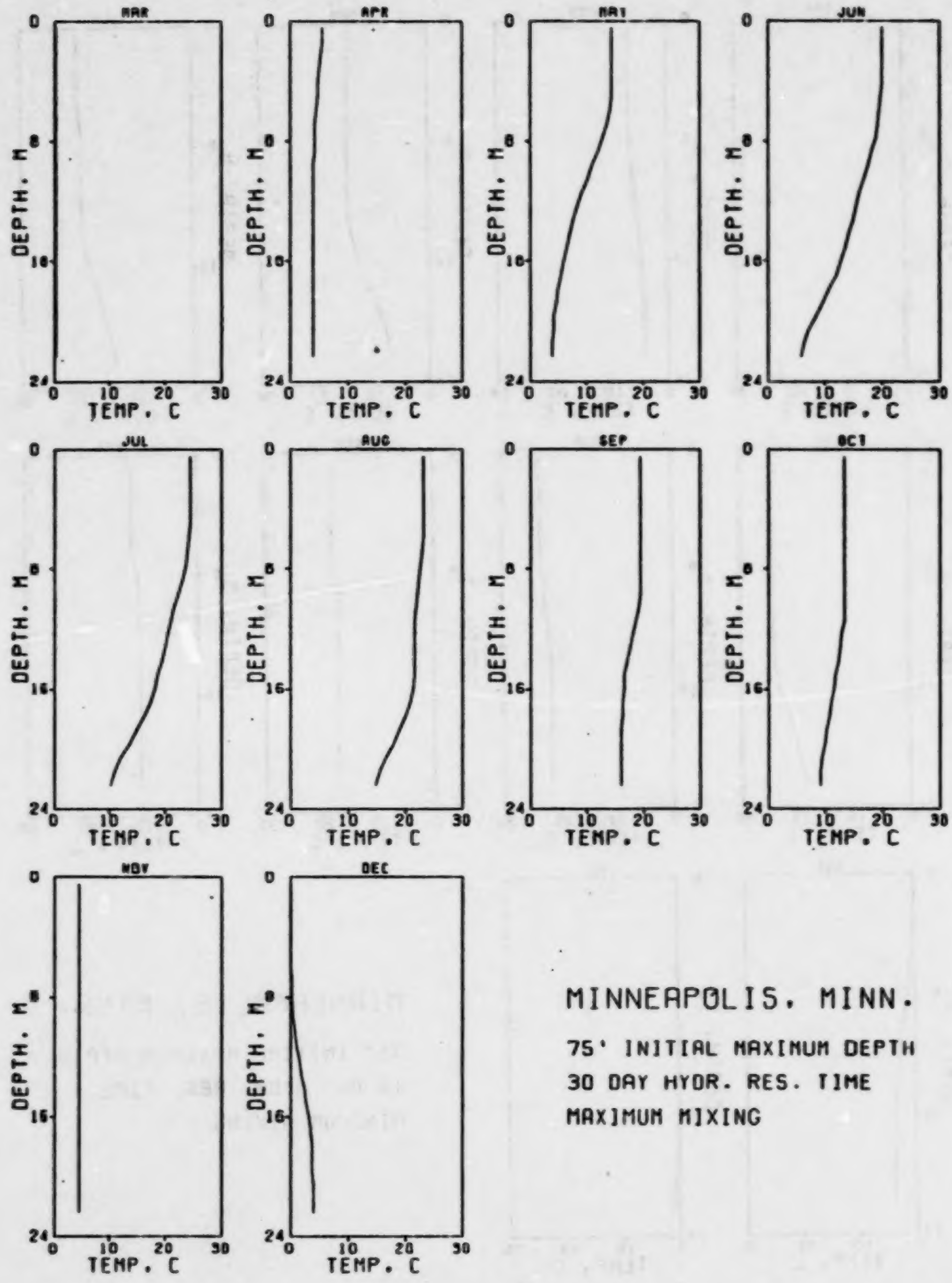
MINNEAPOLIS, MINN.

75' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



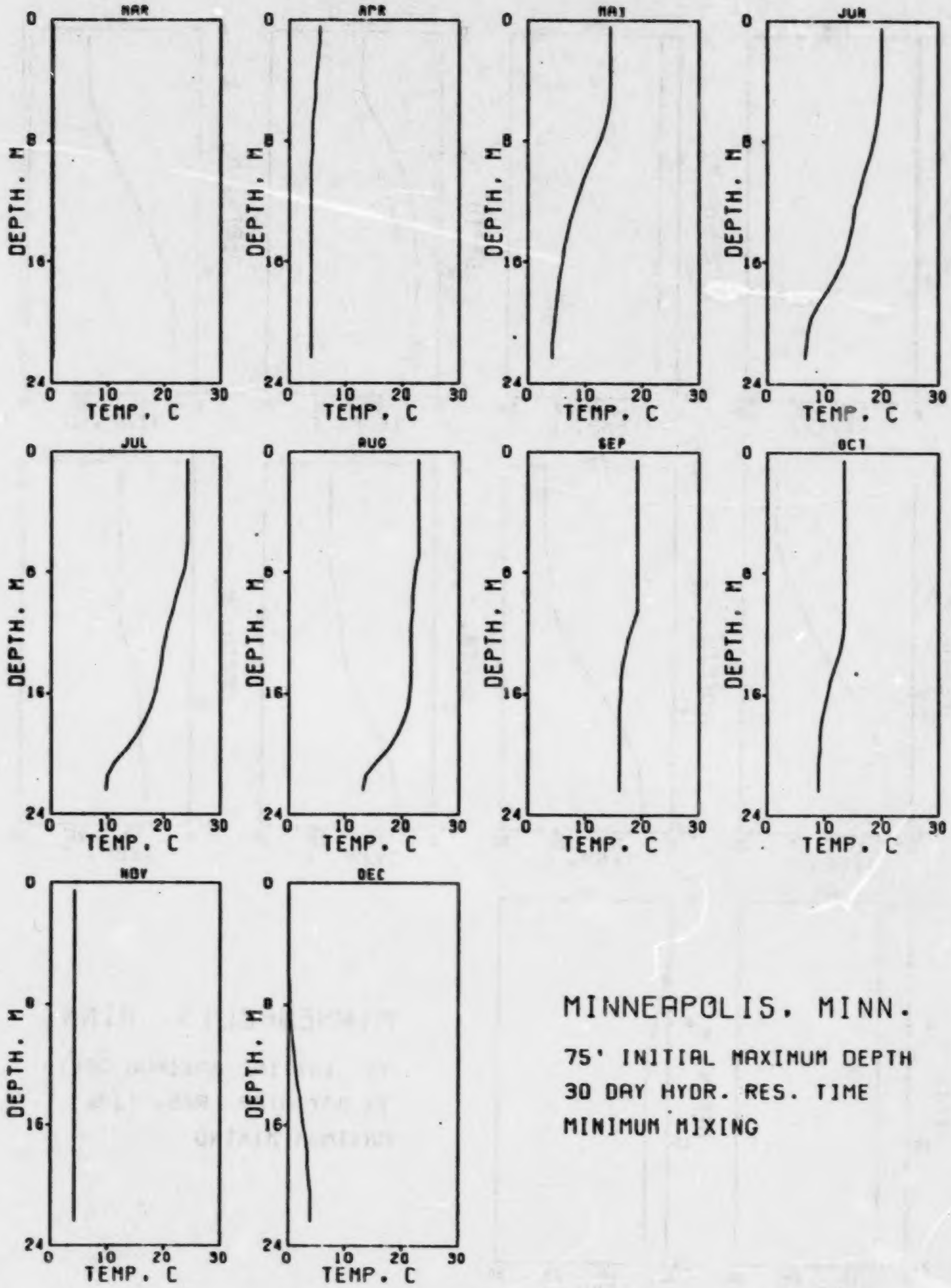
MINNEAPOLIS, MINN.

75' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



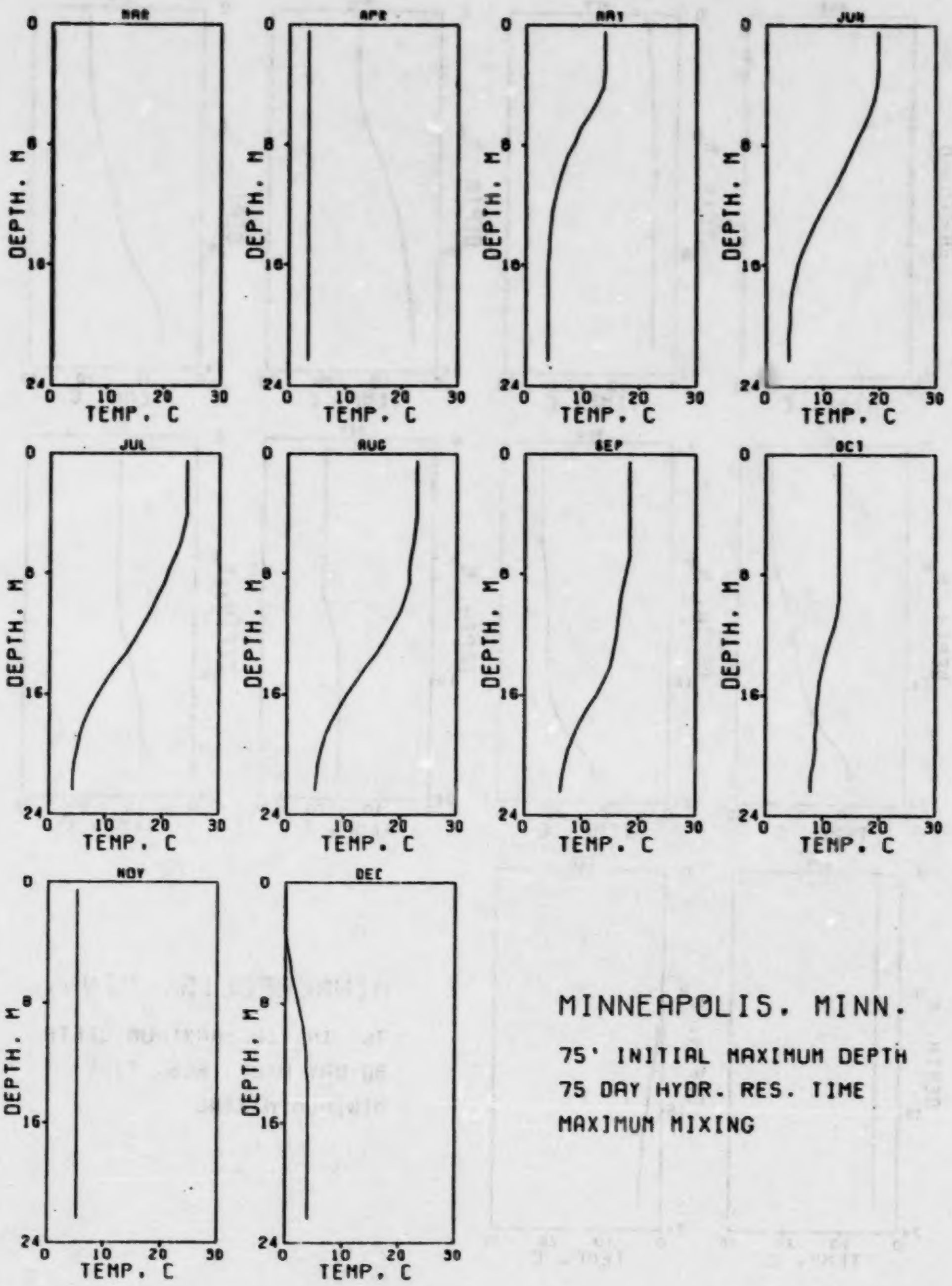
MINNEAPOLIS, MINN.

75' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



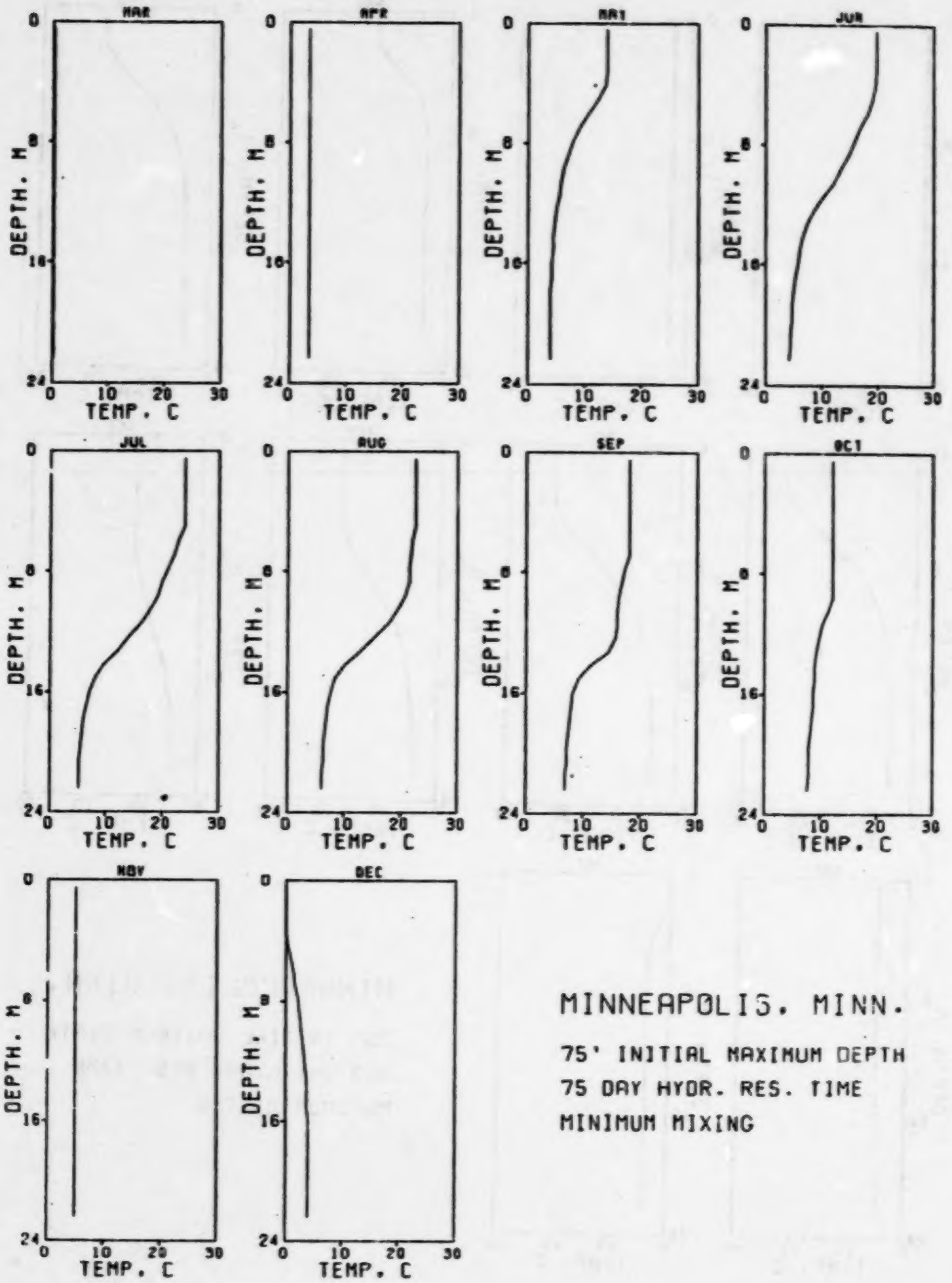
MINNEAPOLIS, MINN.

75' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



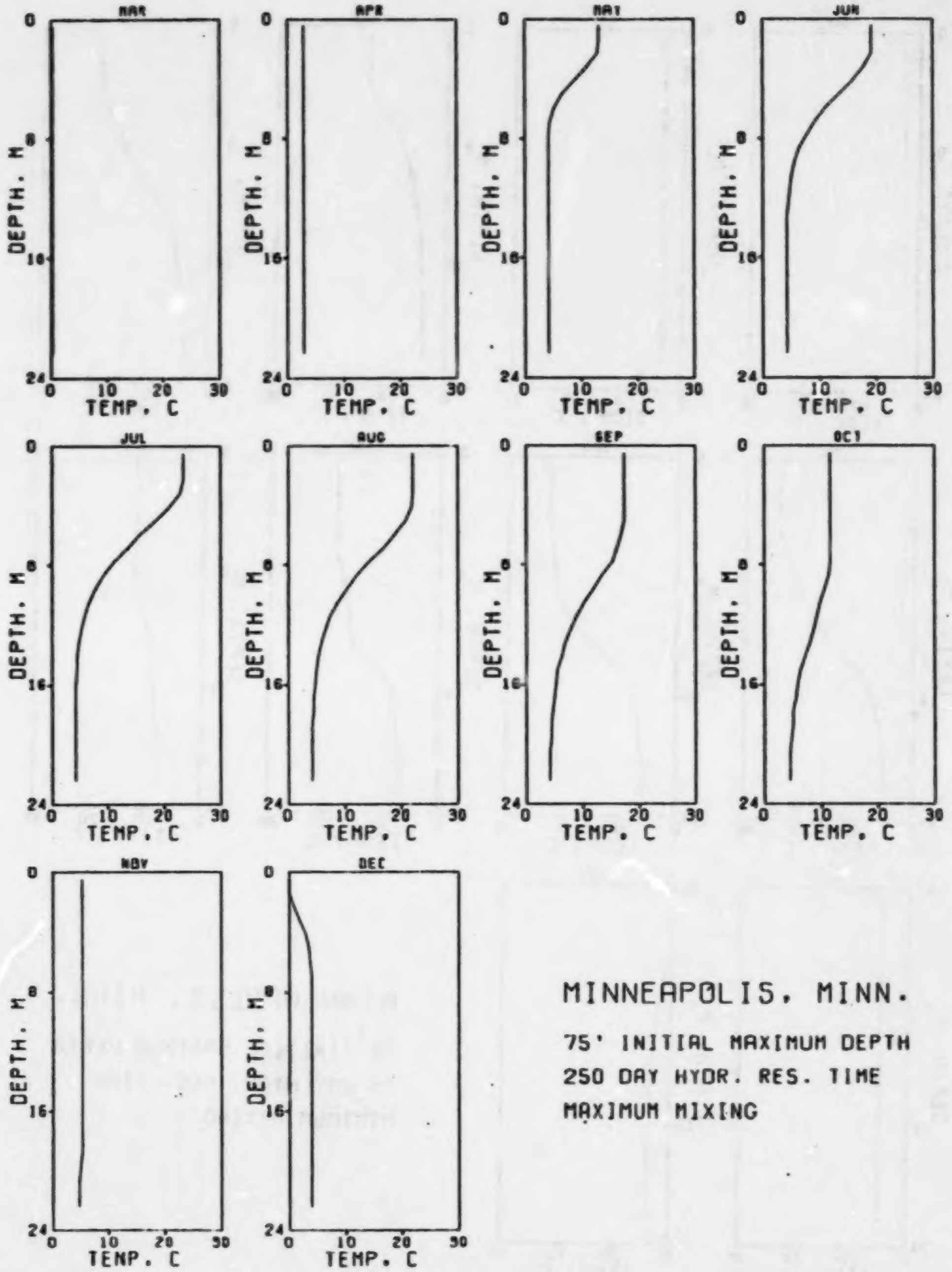
MINNEAPOLIS, MINN.

75' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



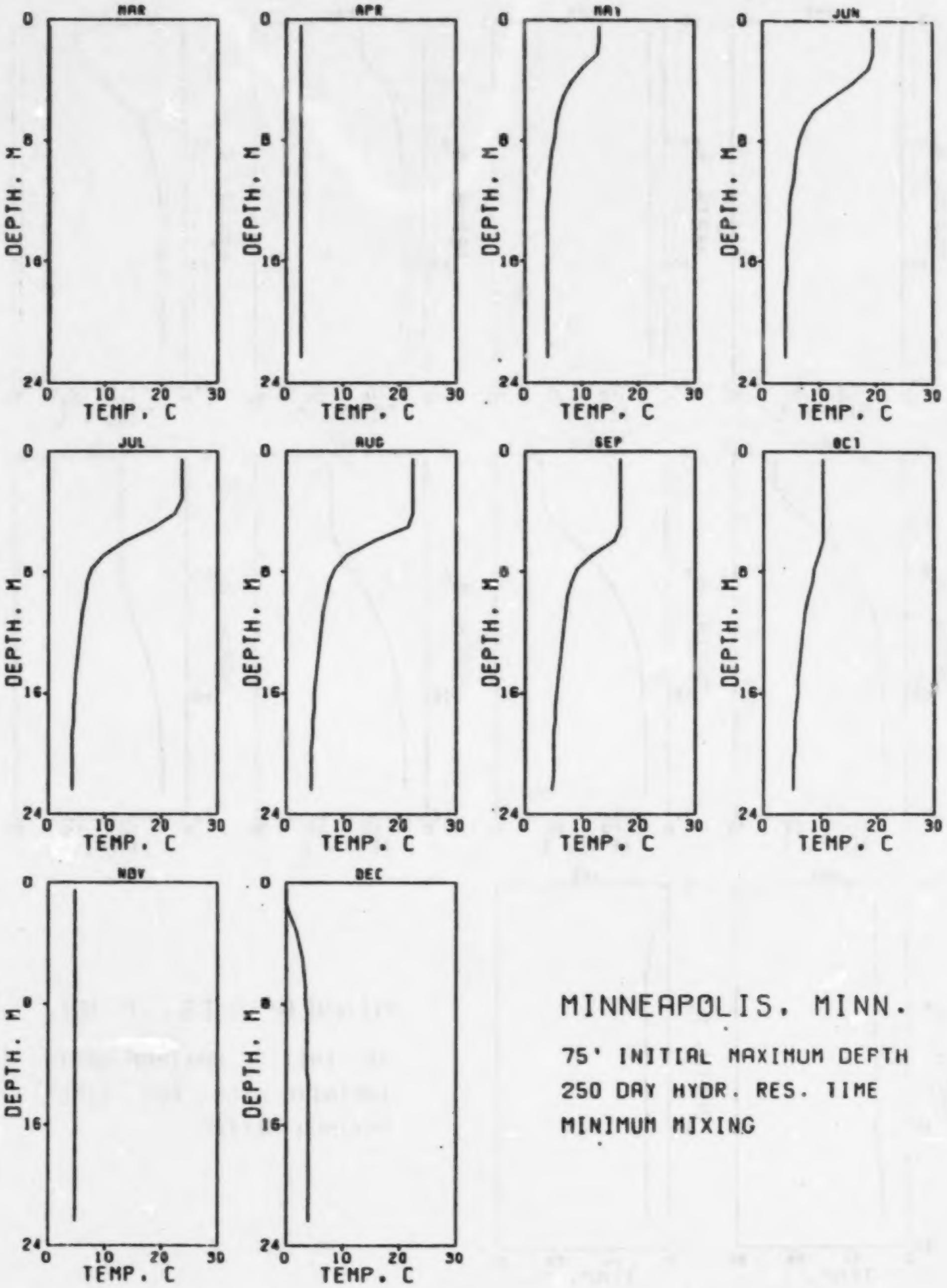
MINNEAPOLIS, MINN.

75' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



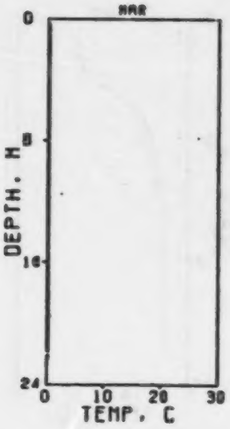
MINNEAPOLIS, MINN.

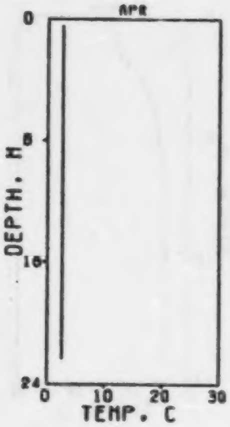
75' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING

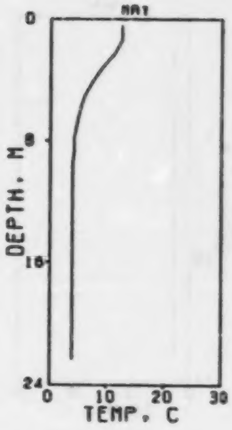


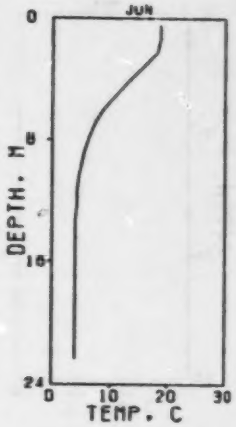
MINNEAPOLIS, MINN.

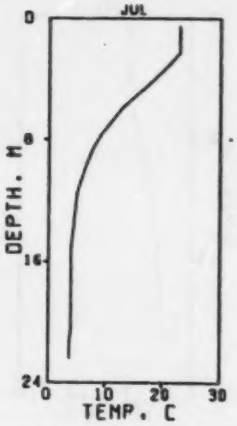
75' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING

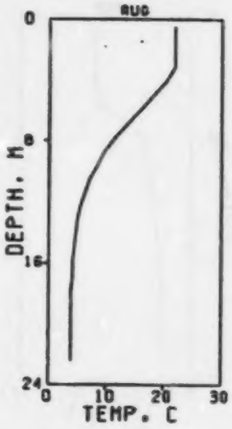


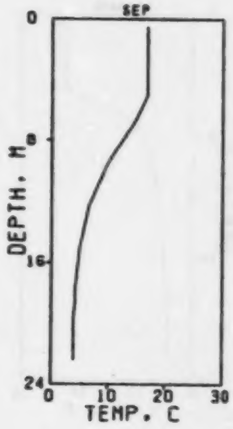


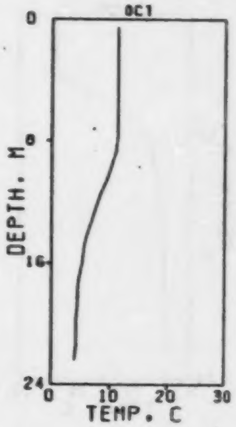


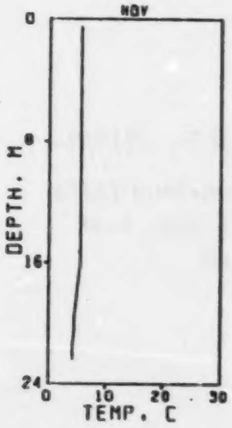


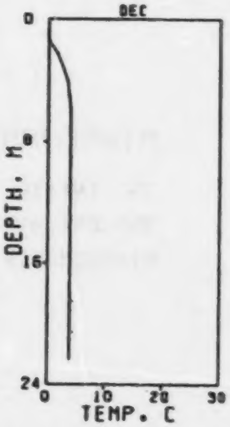






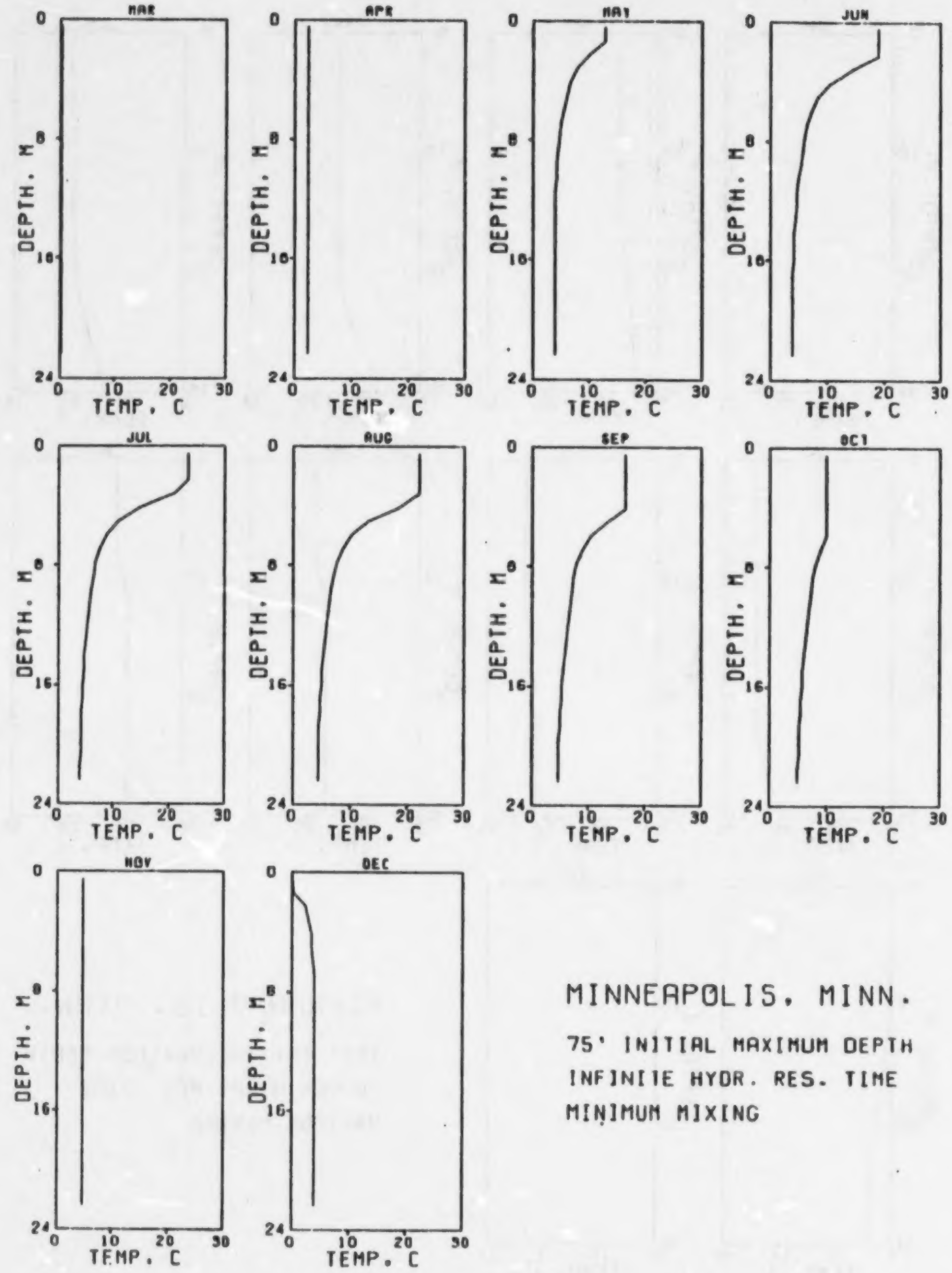






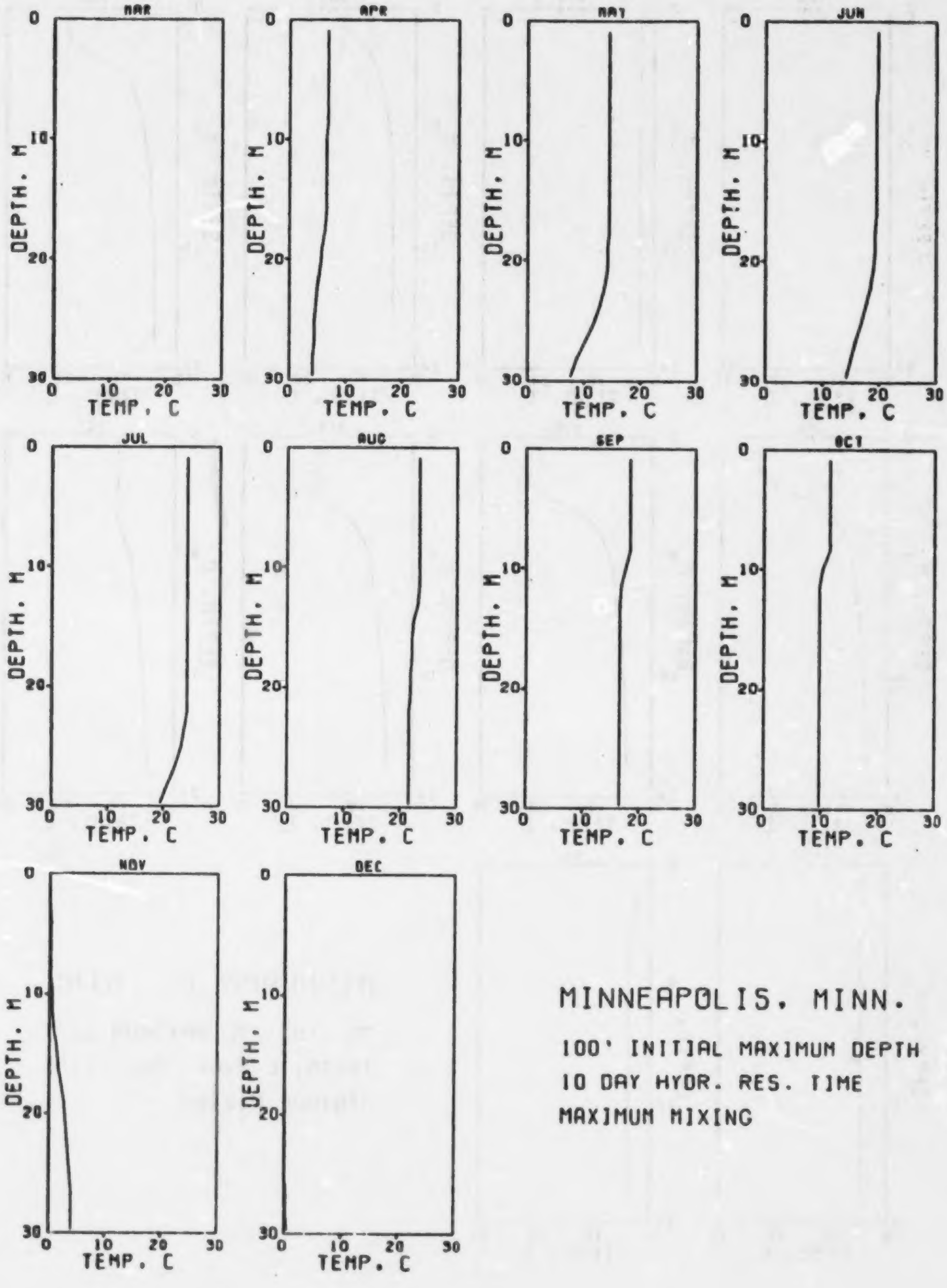
MINNEAPOLIS, MINN.

75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



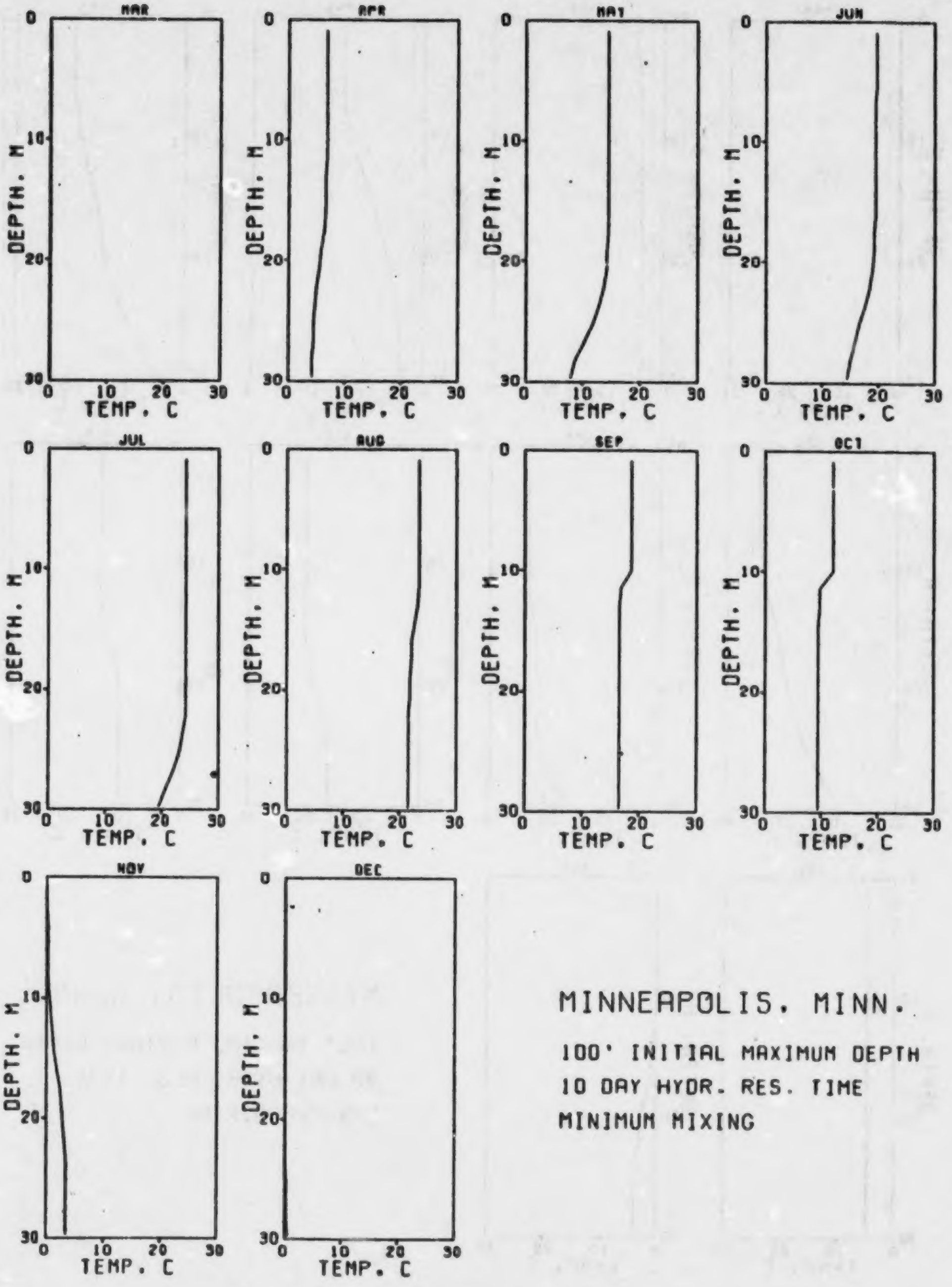
MINNEAPOLIS, MINN.

75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



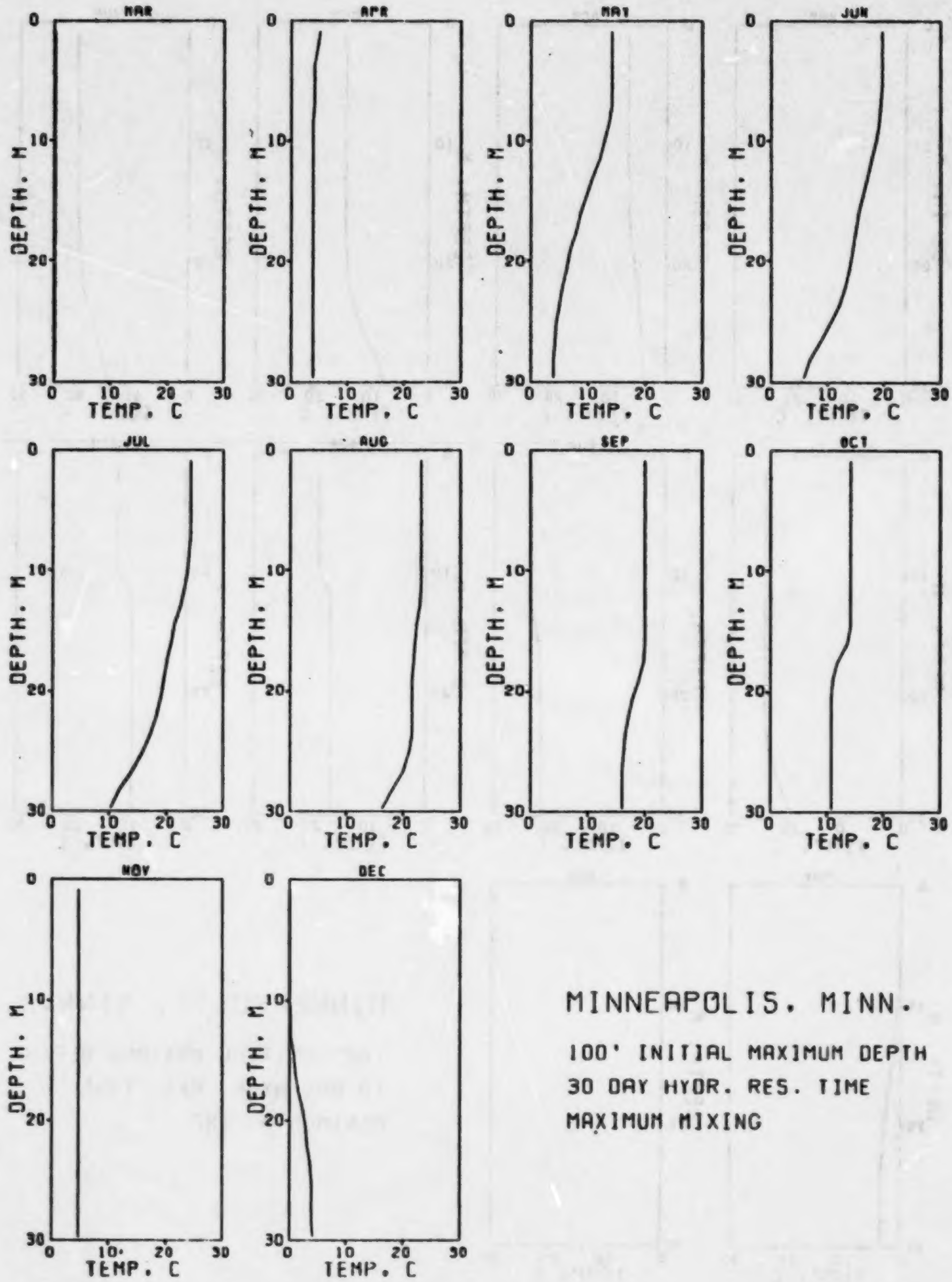
MINNEAPOLIS, MINN.

100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



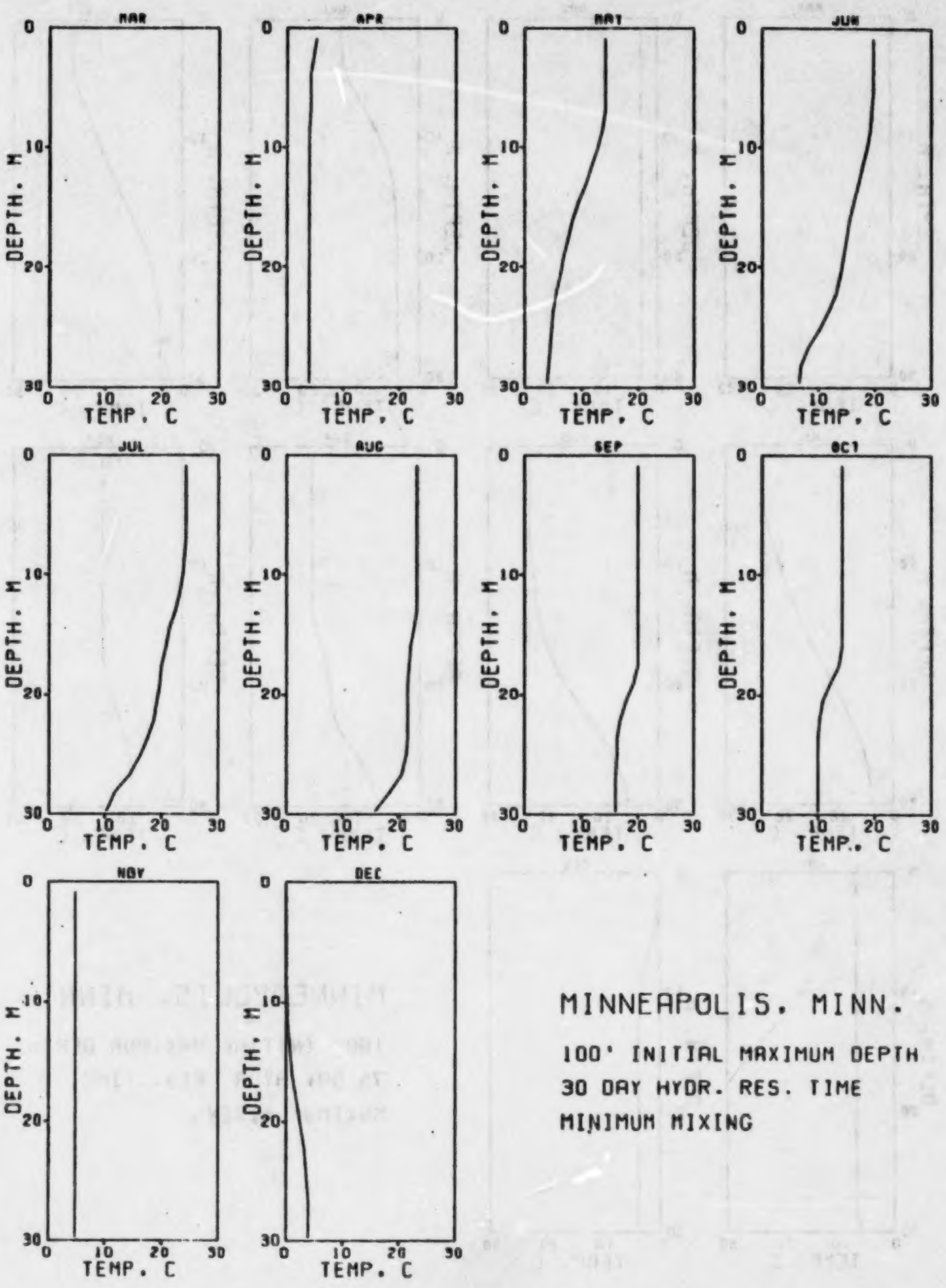
MINNEAPOLIS, MINN.

100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



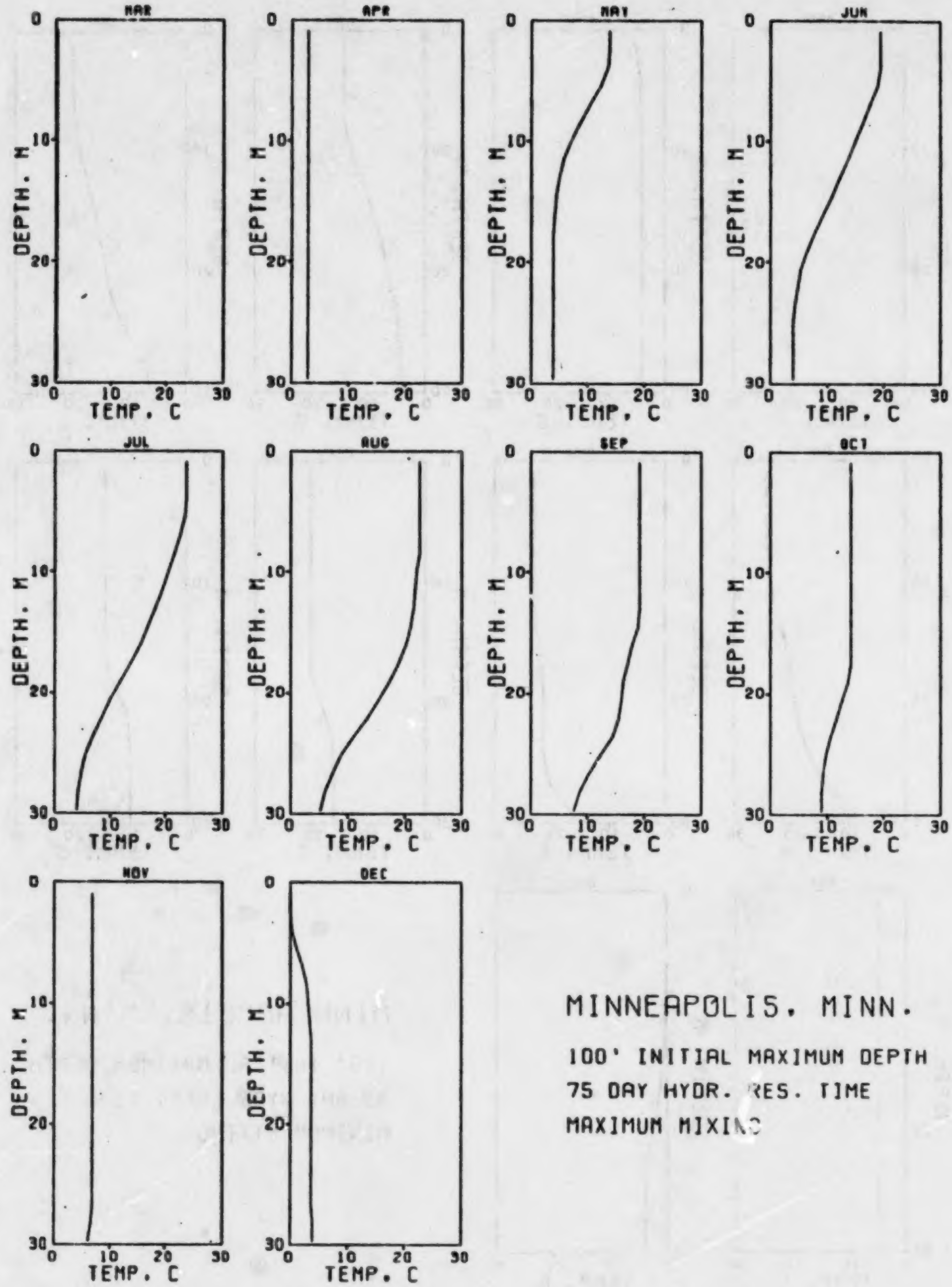
MINNEAPOLIS, MINN.

100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



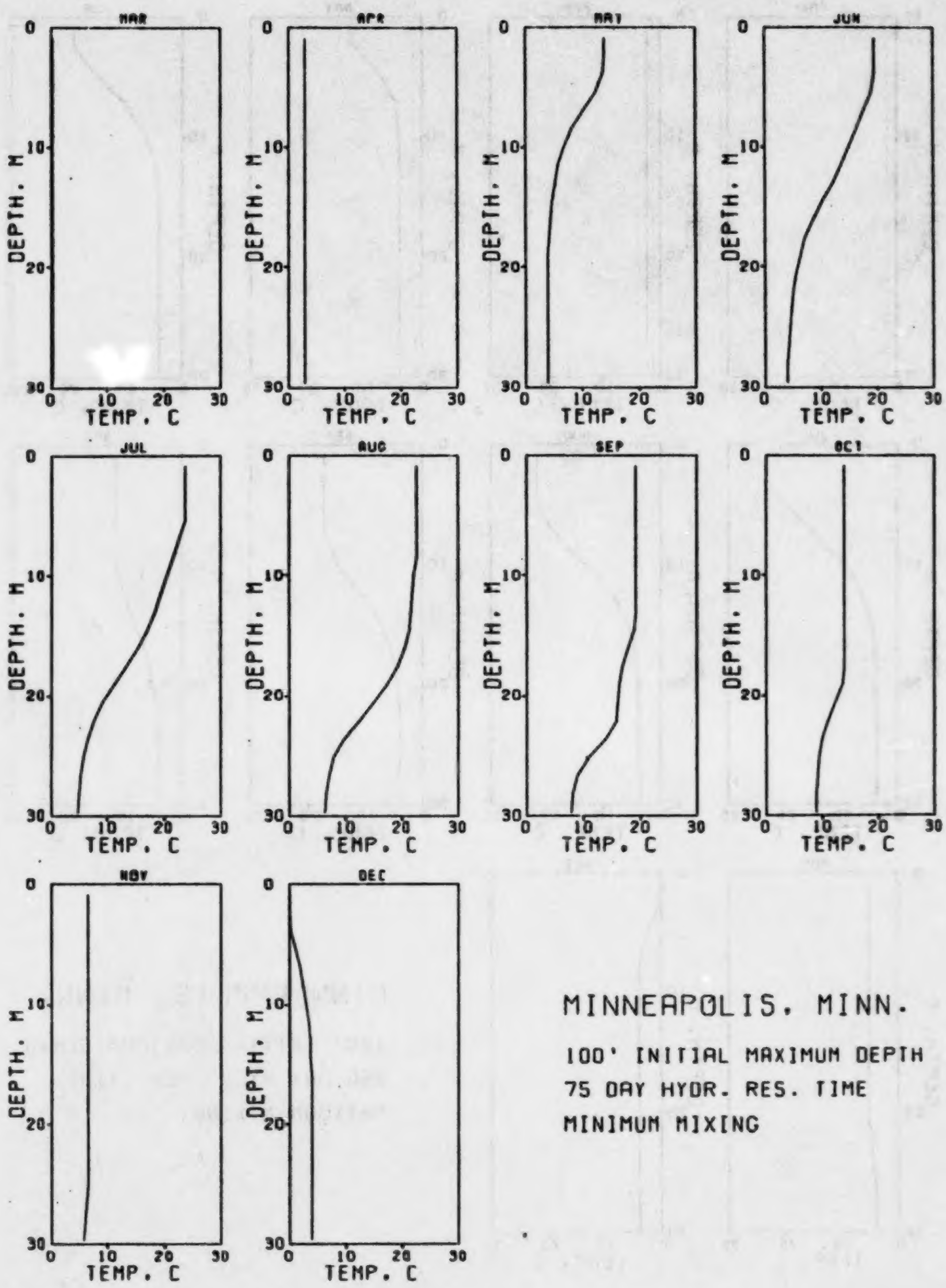
MINNEAPOLIS. MINN.

100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



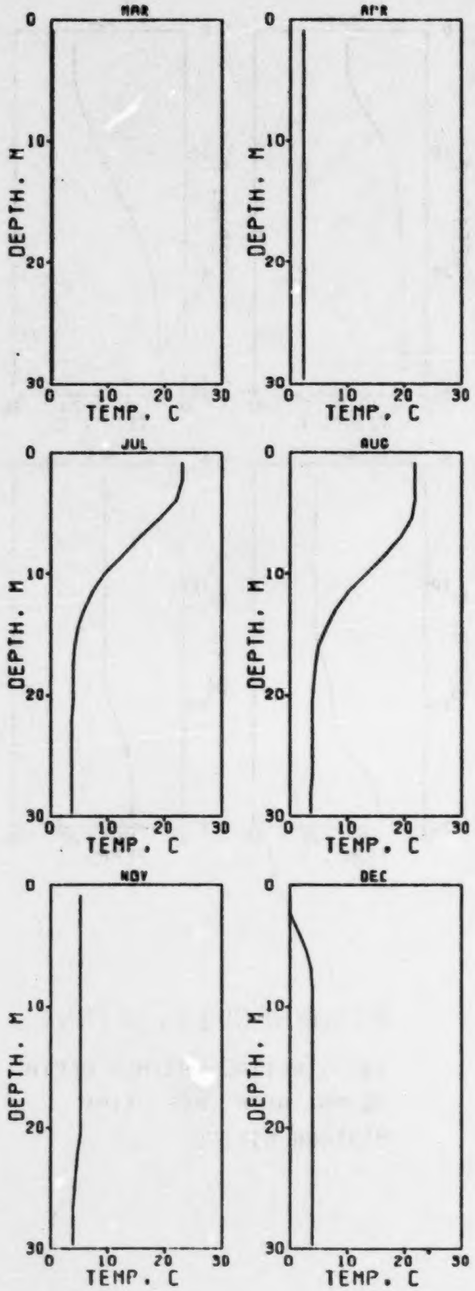
MINNEAPOLIS, MINN.

100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



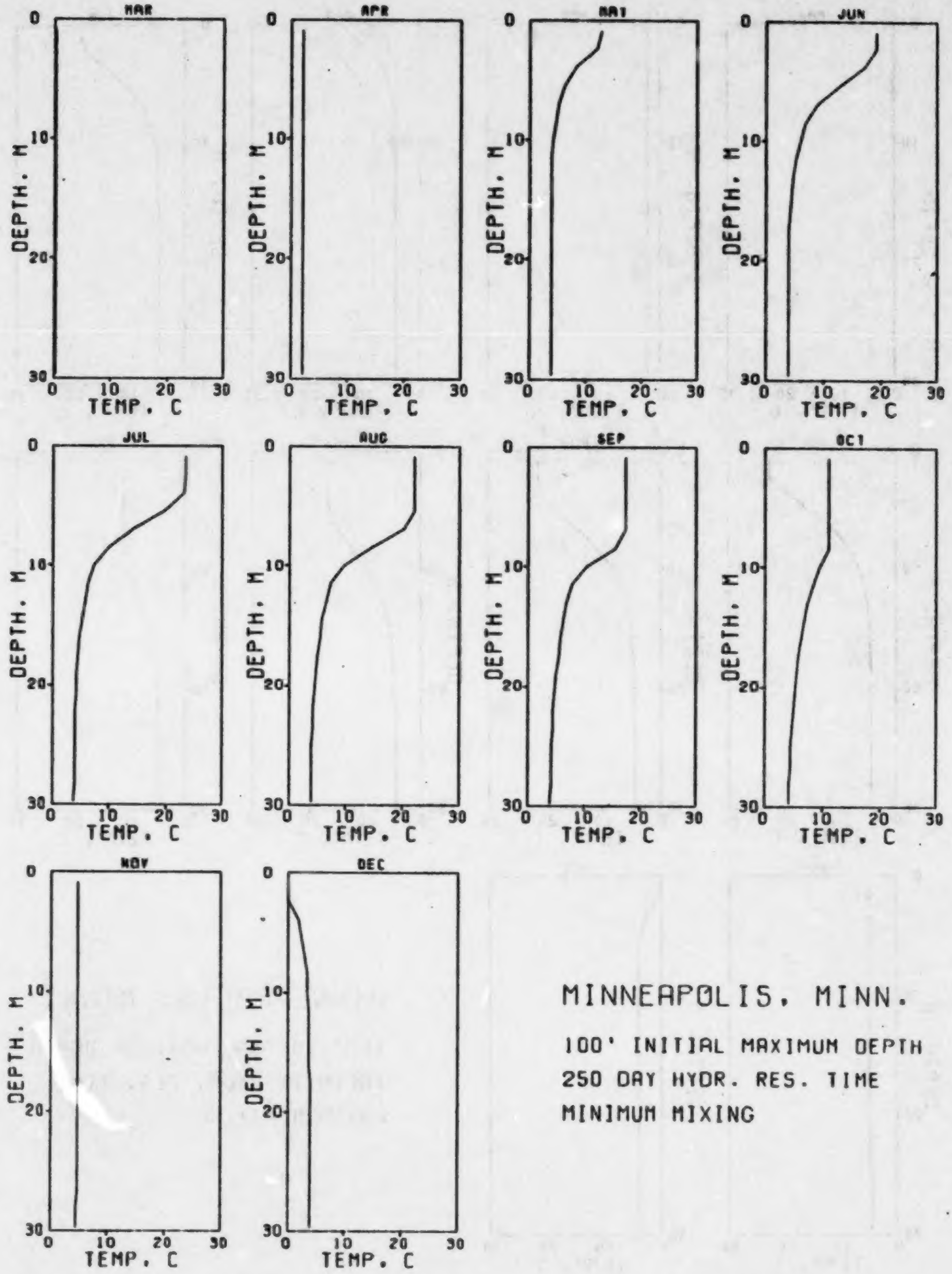
MINNEAPOLIS, MINN.

100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



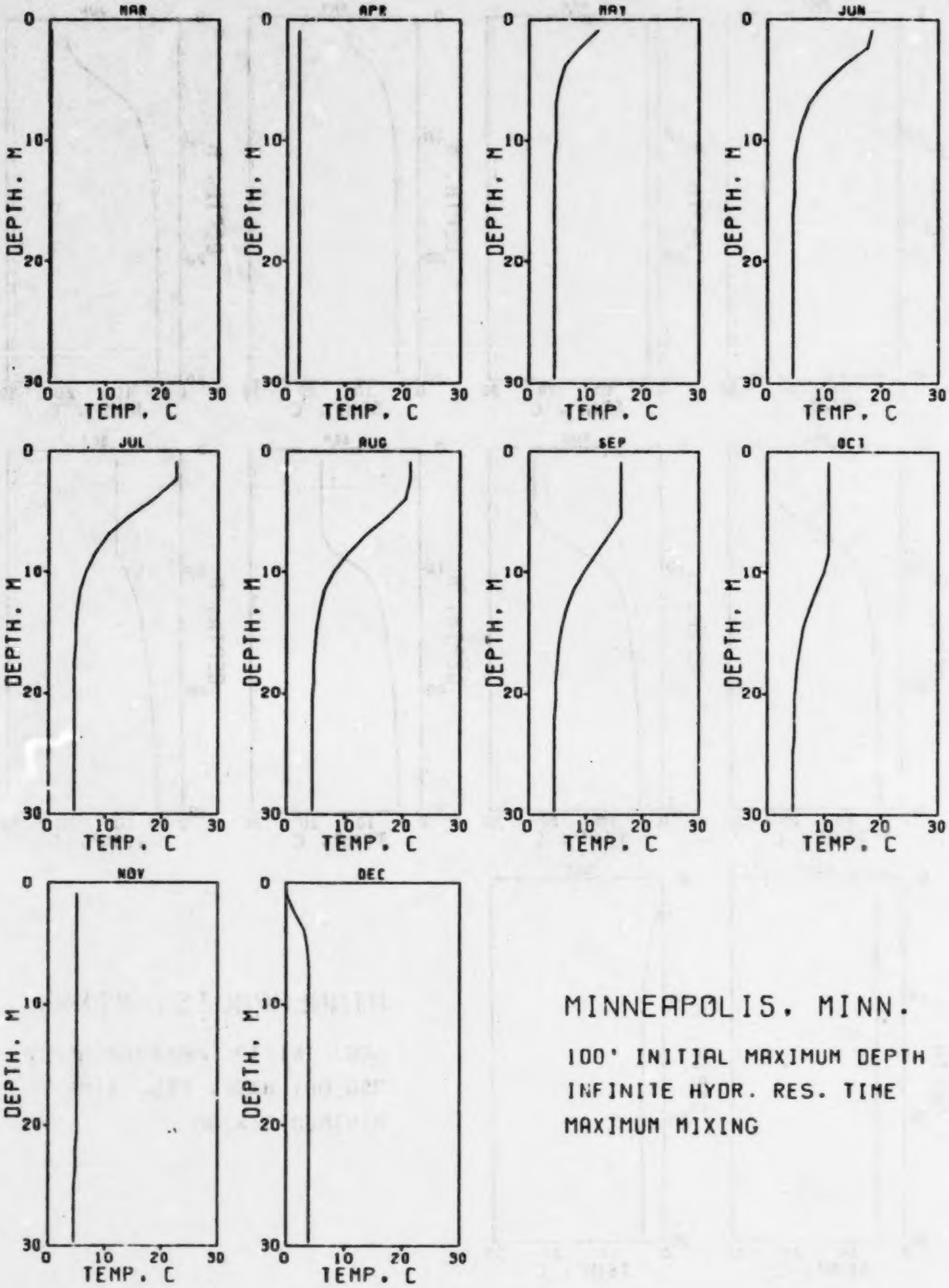
MINNEAPOLIS, MINN.

100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



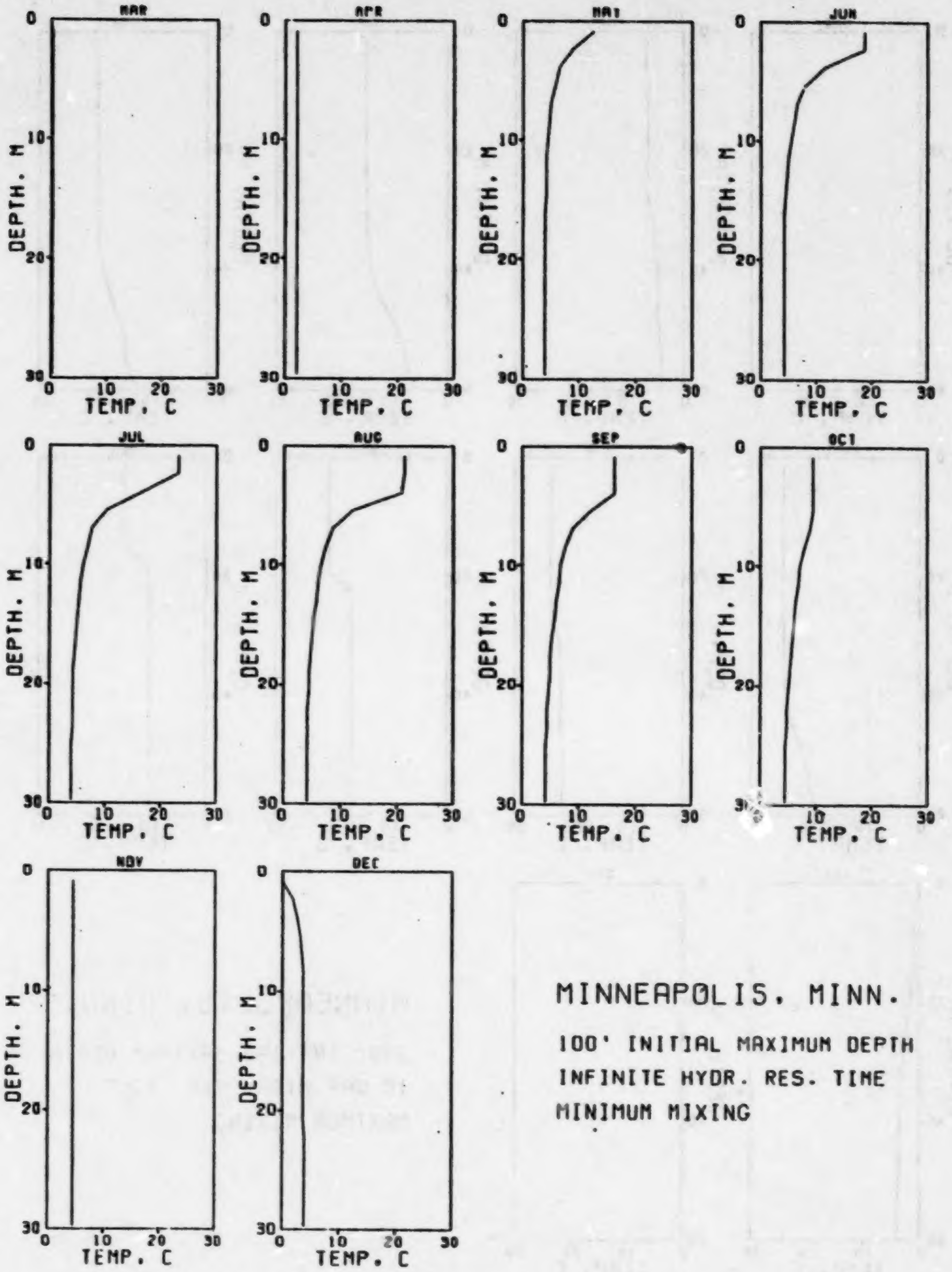
MINNEAPOLIS, MINN.

100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



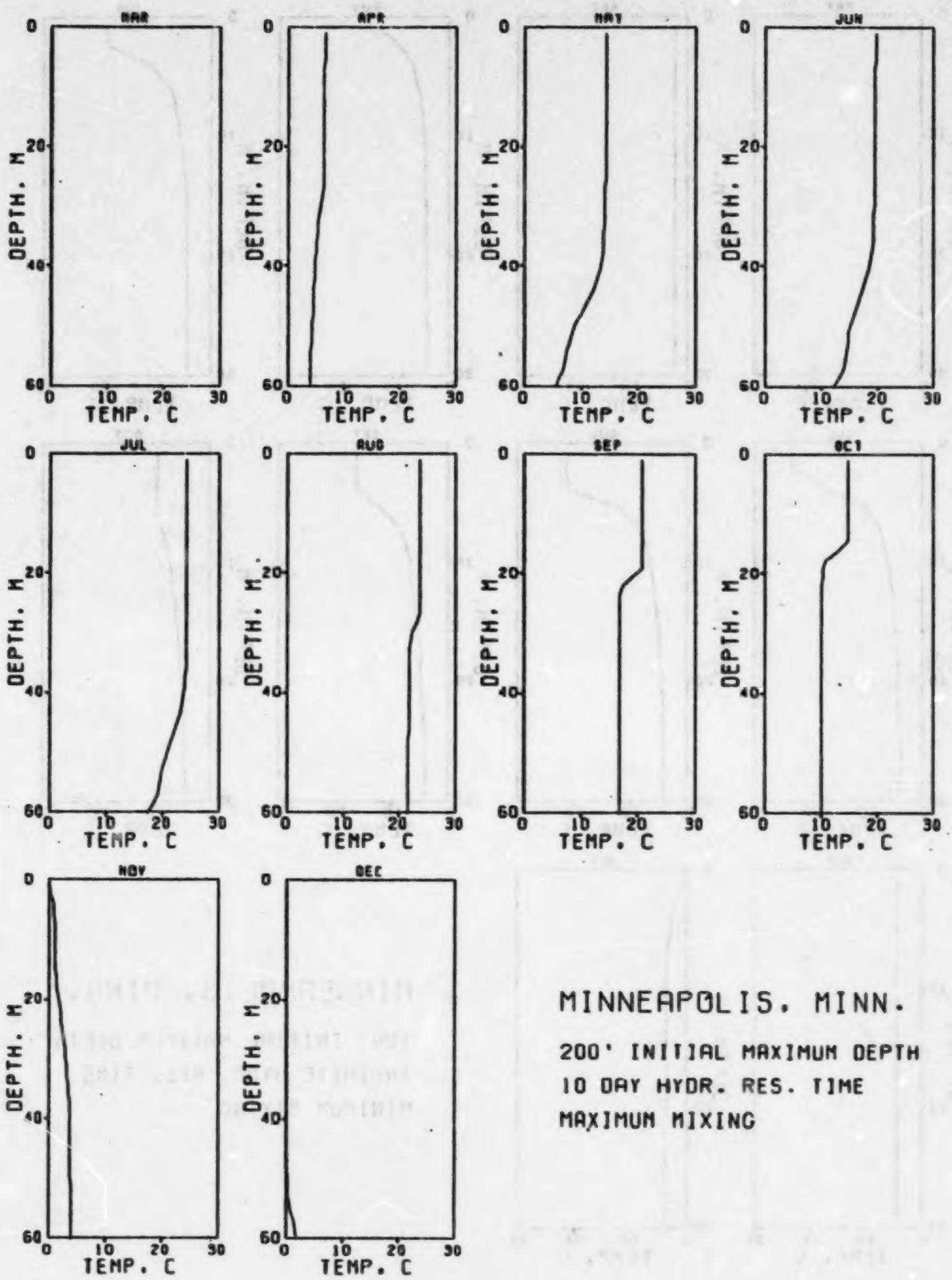
MINNEAPOLIS, MINN.

100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



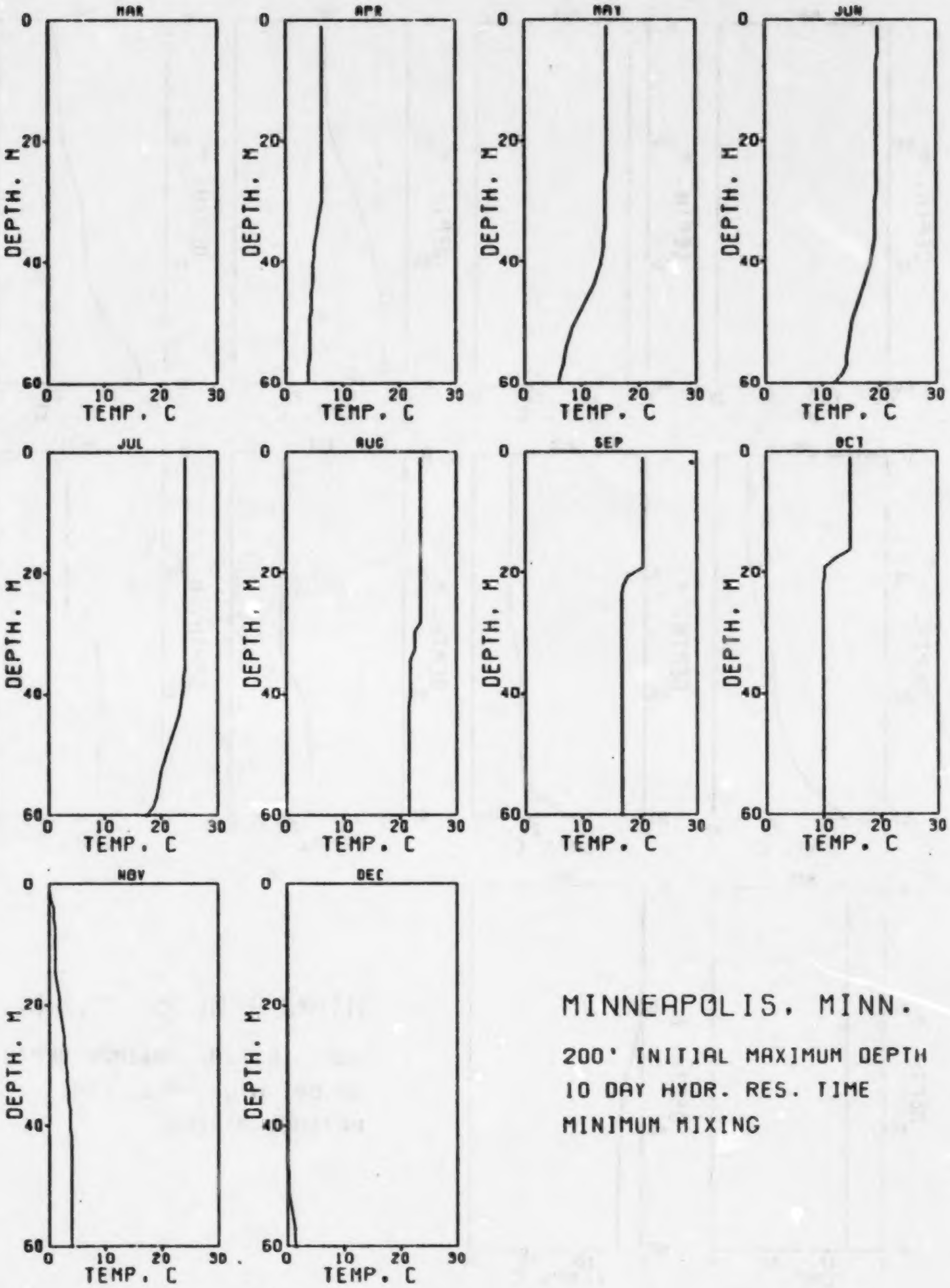
MINNEAPOLIS, MINN.

100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



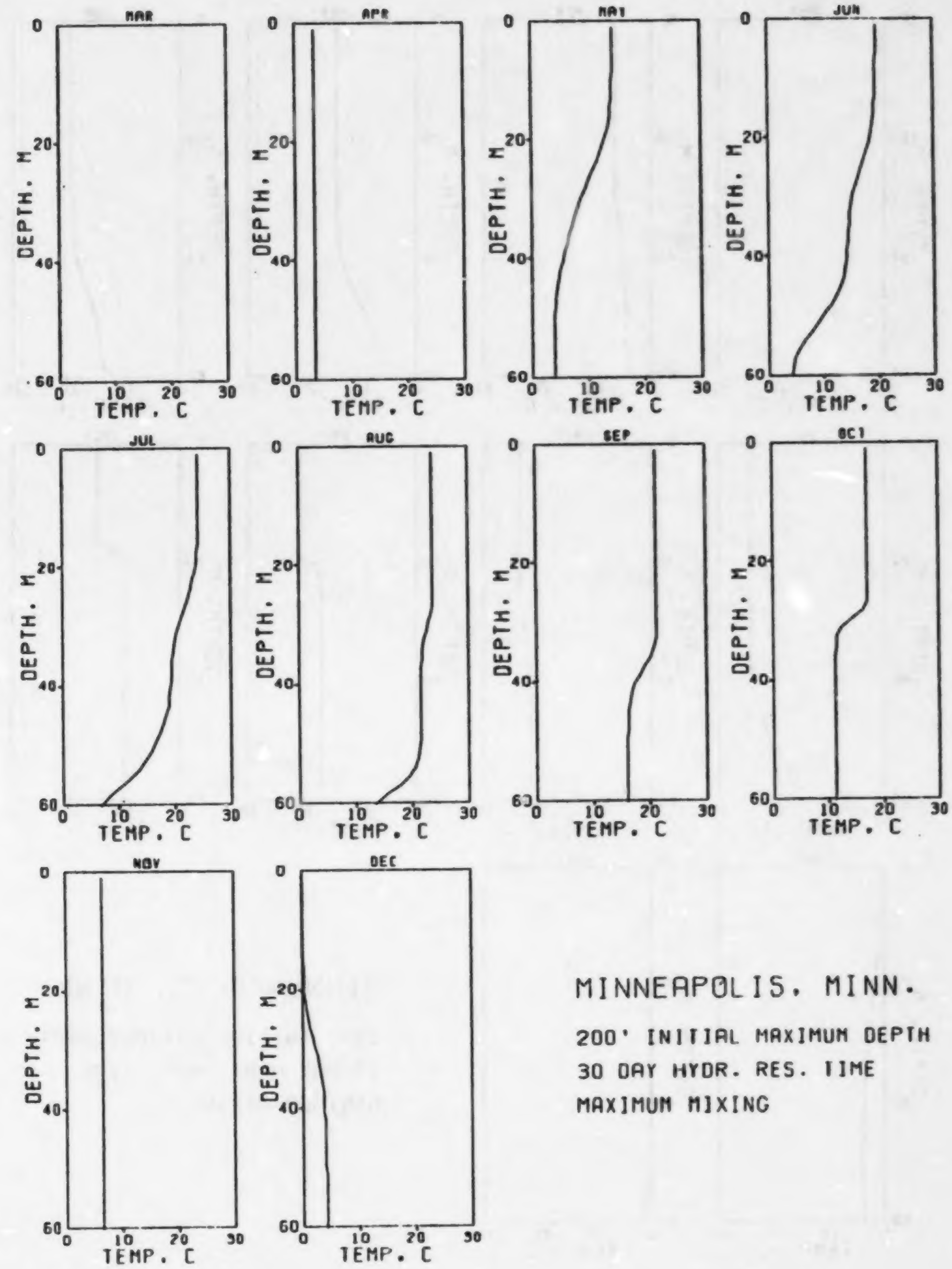
MINNEAPOLIS, MINN.

200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



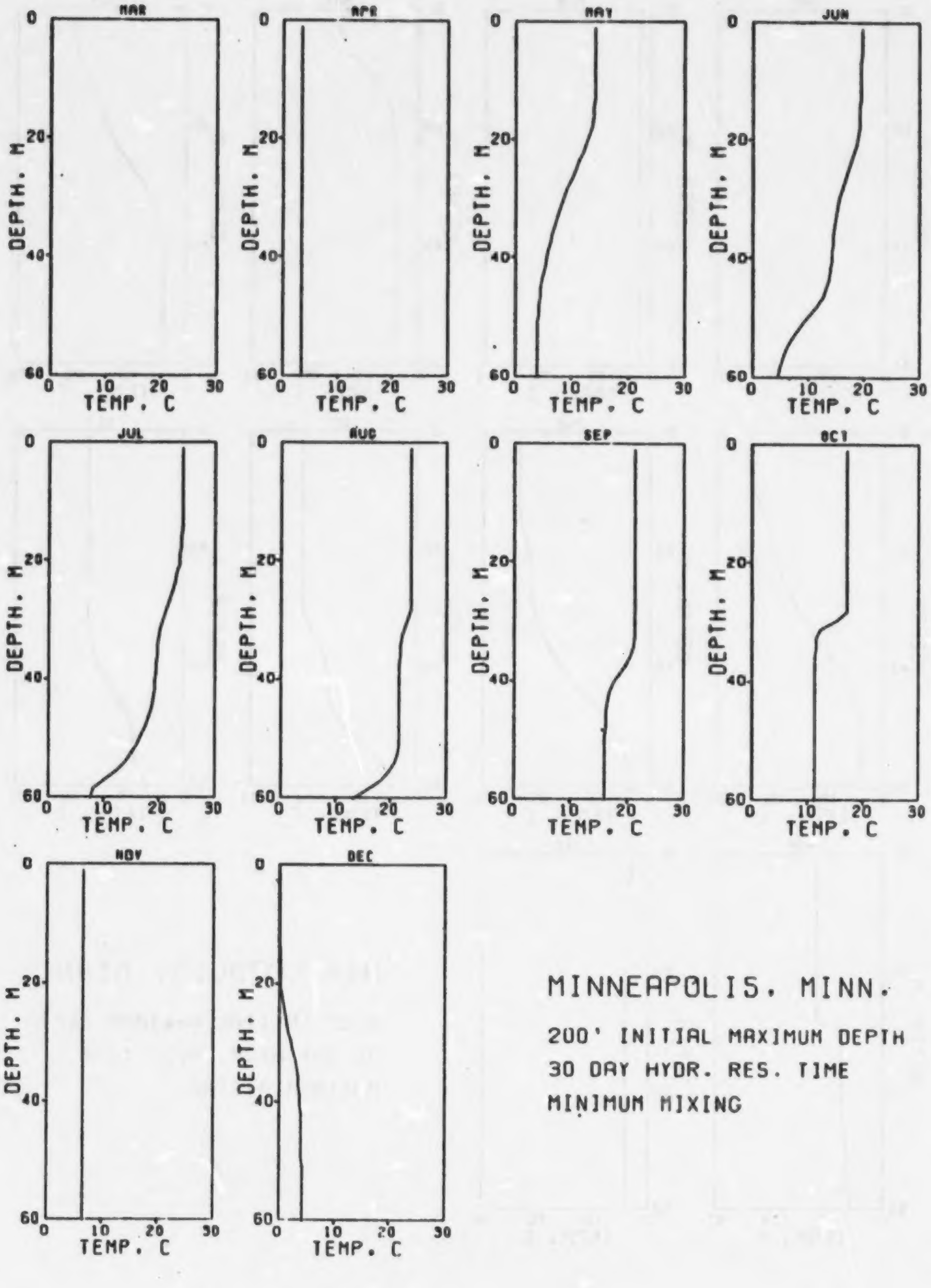
MINNEAPOLIS, MINN.

200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



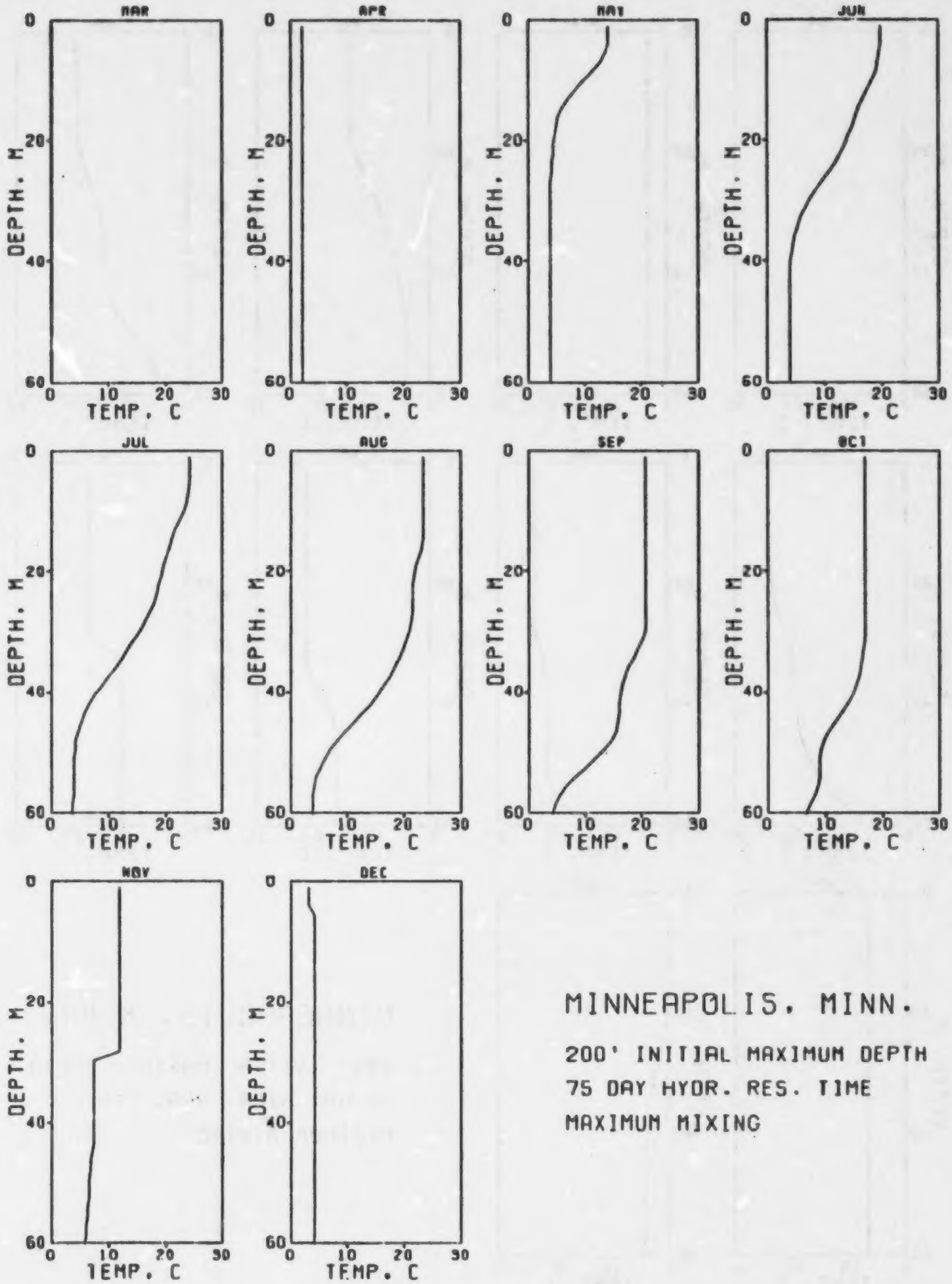
MINNEAPOLIS, MINN.

200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



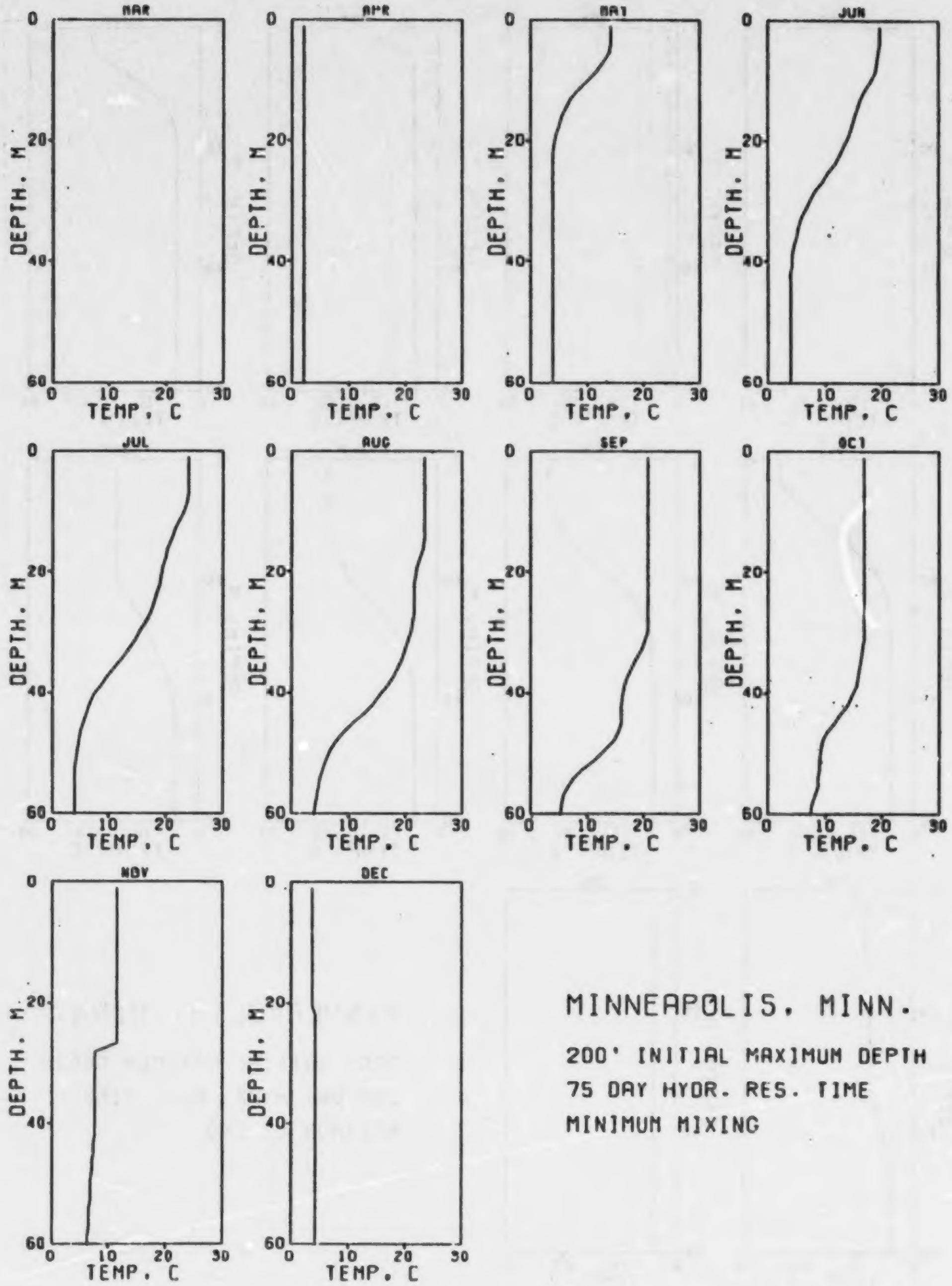
MINNEAPOLIS, MINN.

200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



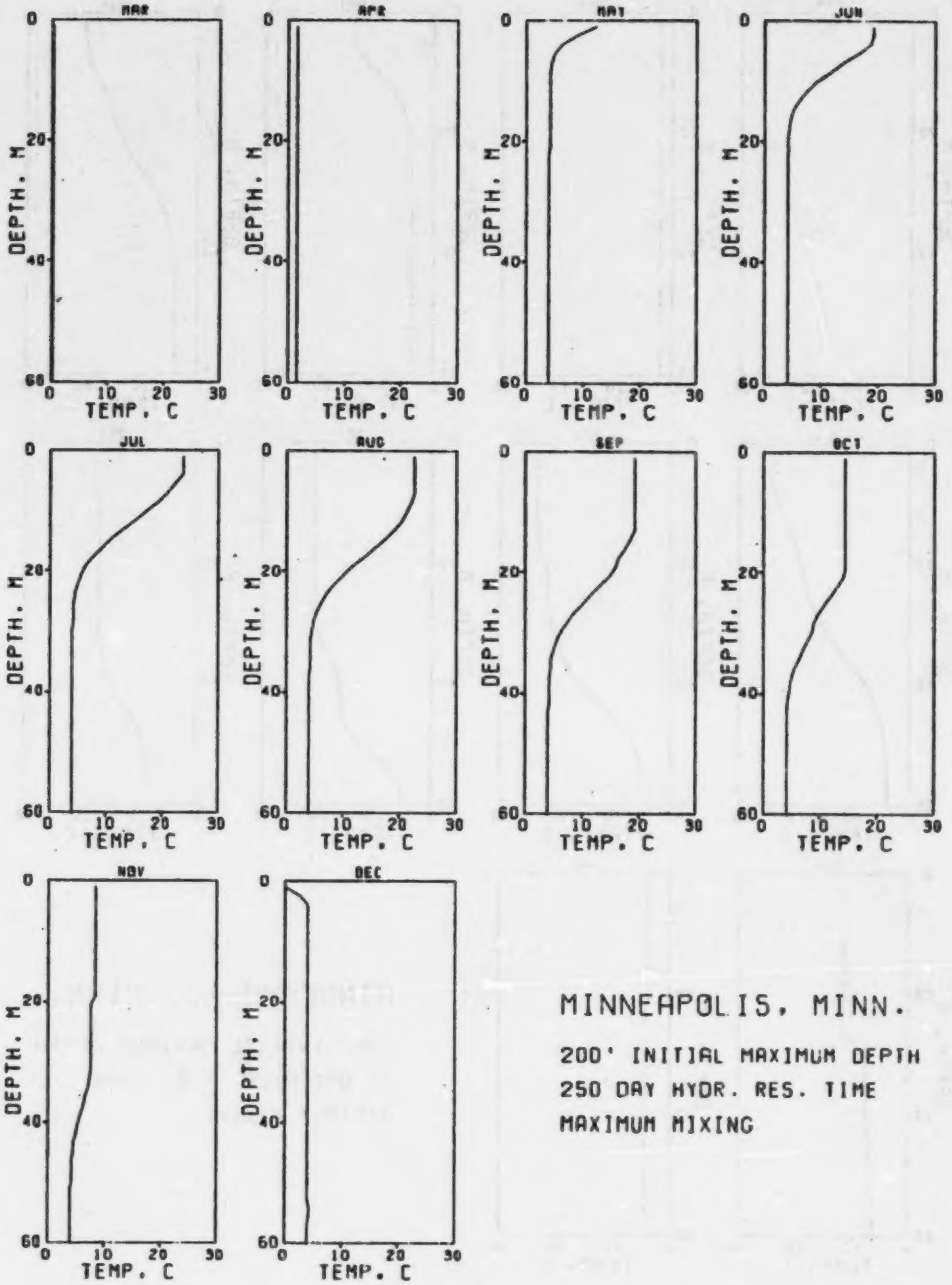
MINNEAPOLIS, MINN.

200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



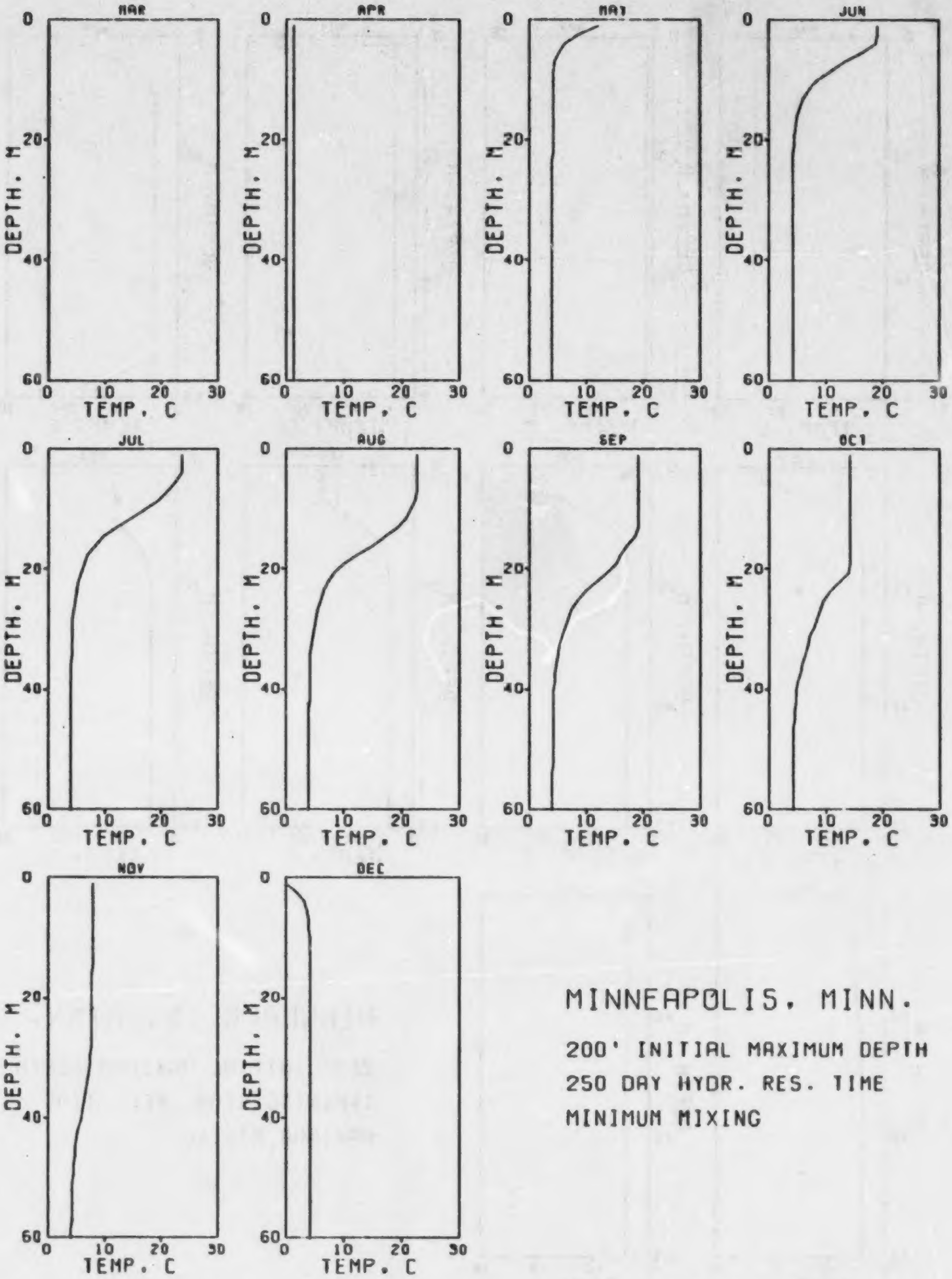
MINNEAPOLIS, MINN.

200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



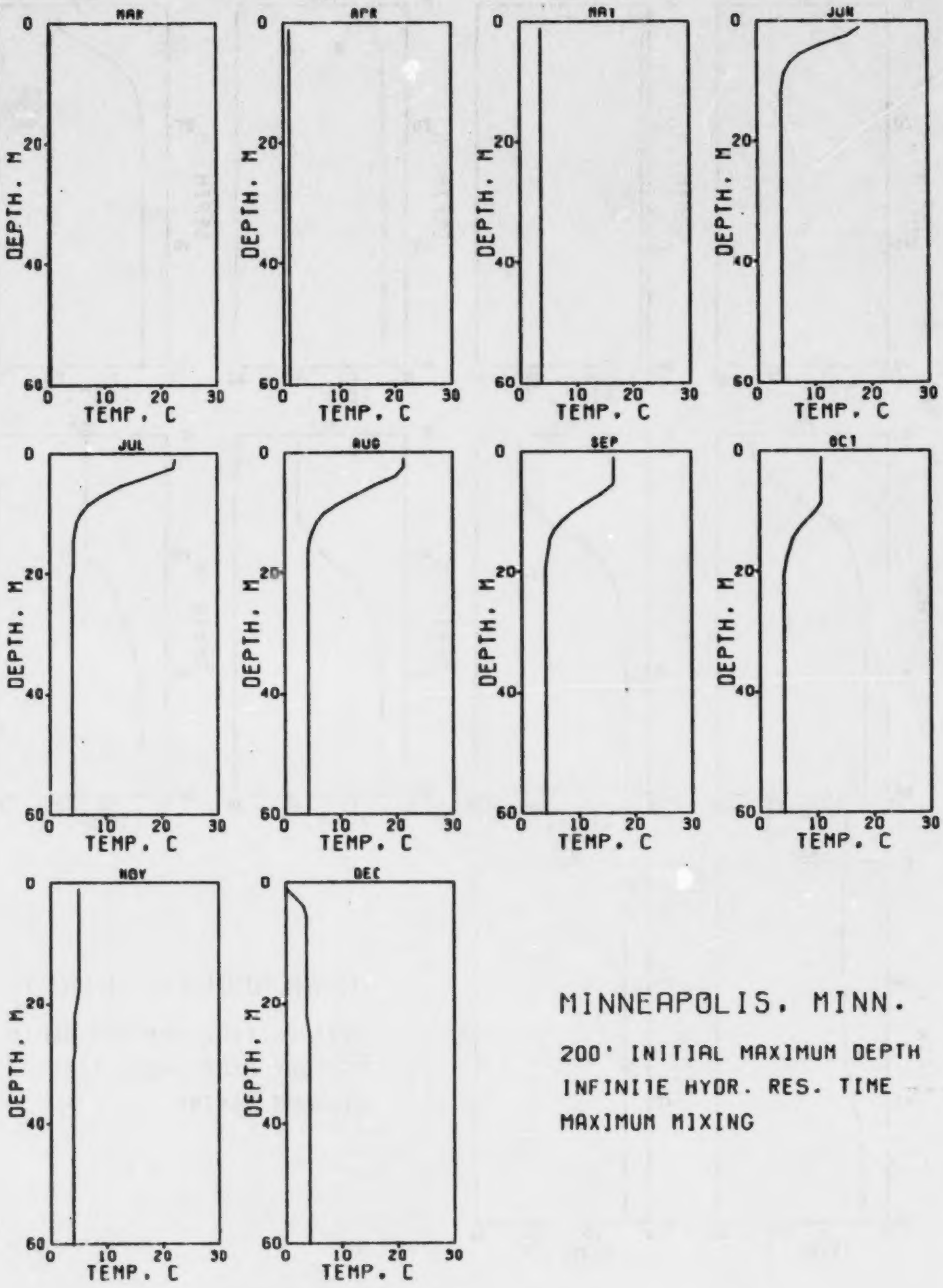
MINNEAPOLIS, MINN.

200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



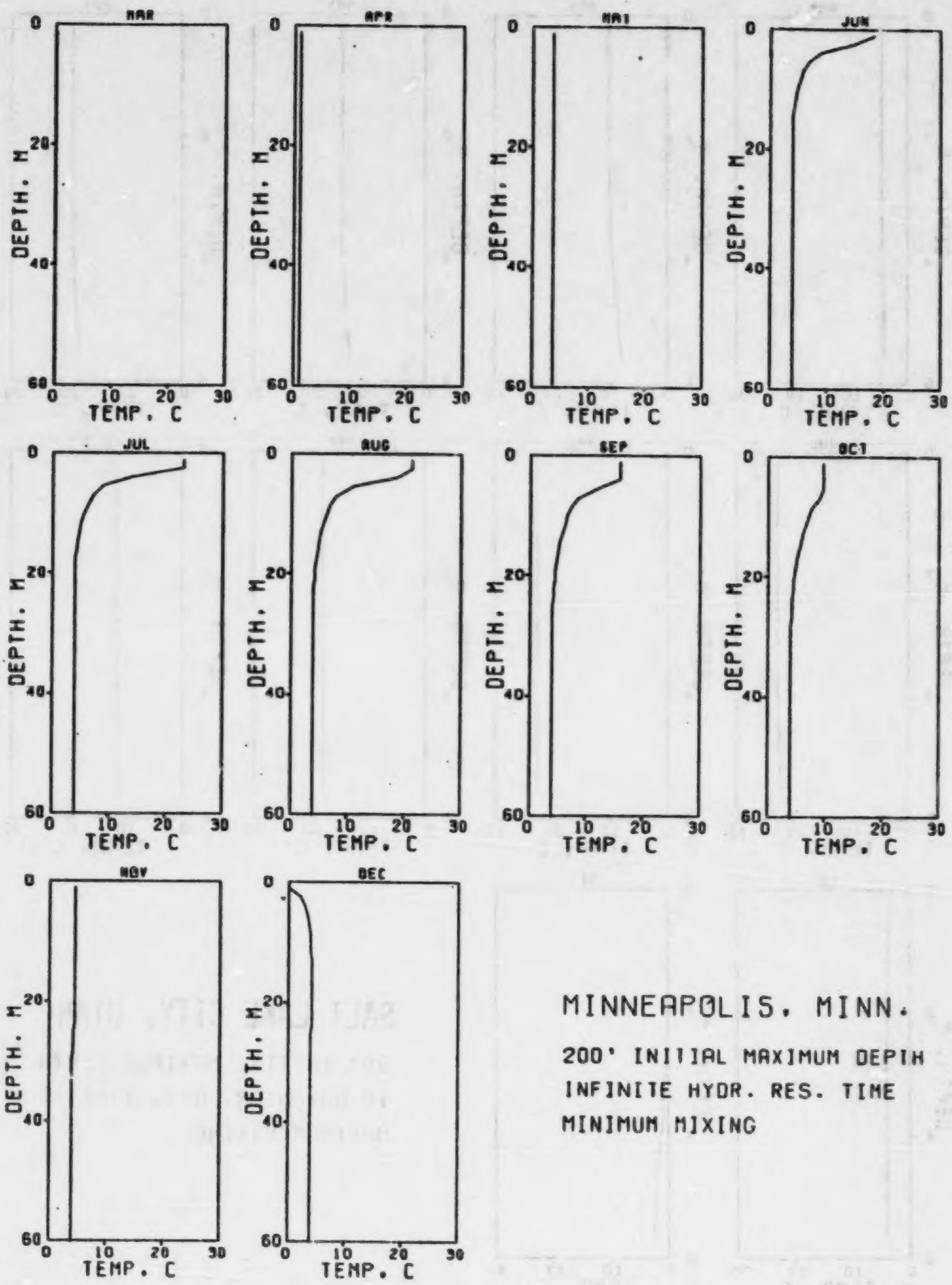
MINNEAPOLIS, MINN.

200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



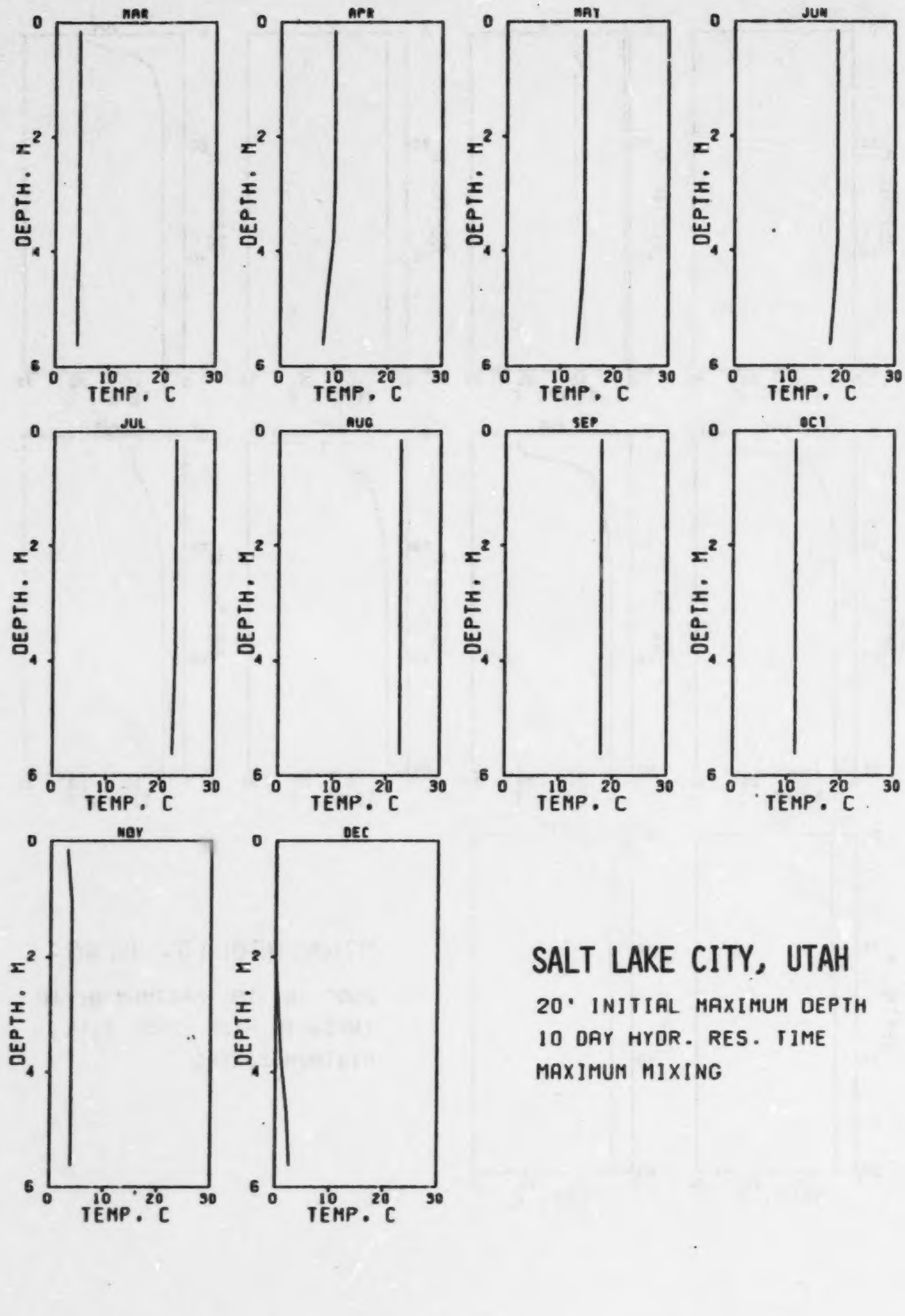
MINNEAPOLIS, MINN.

200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



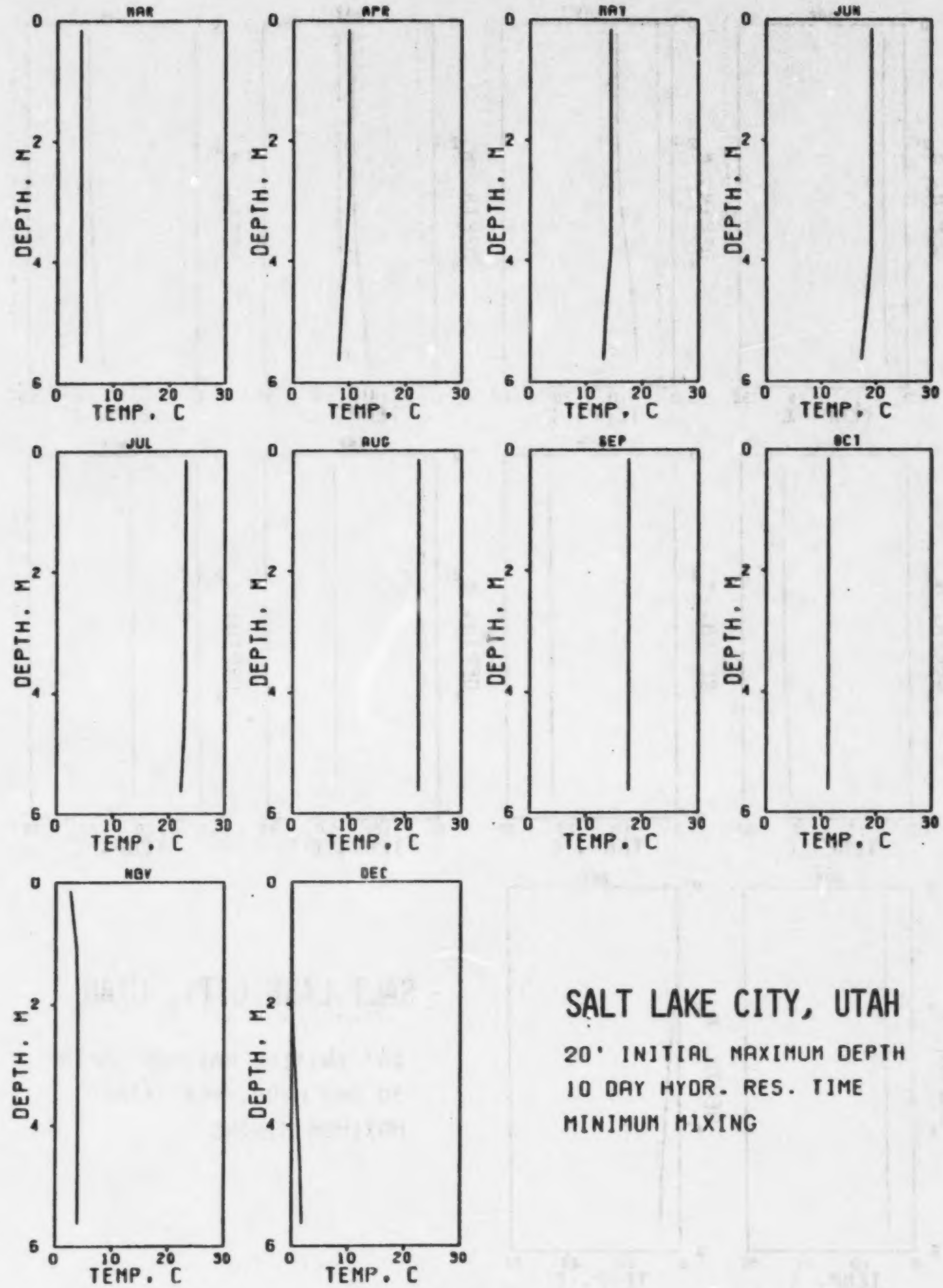
MINNEAPOLIS, MINN.

200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



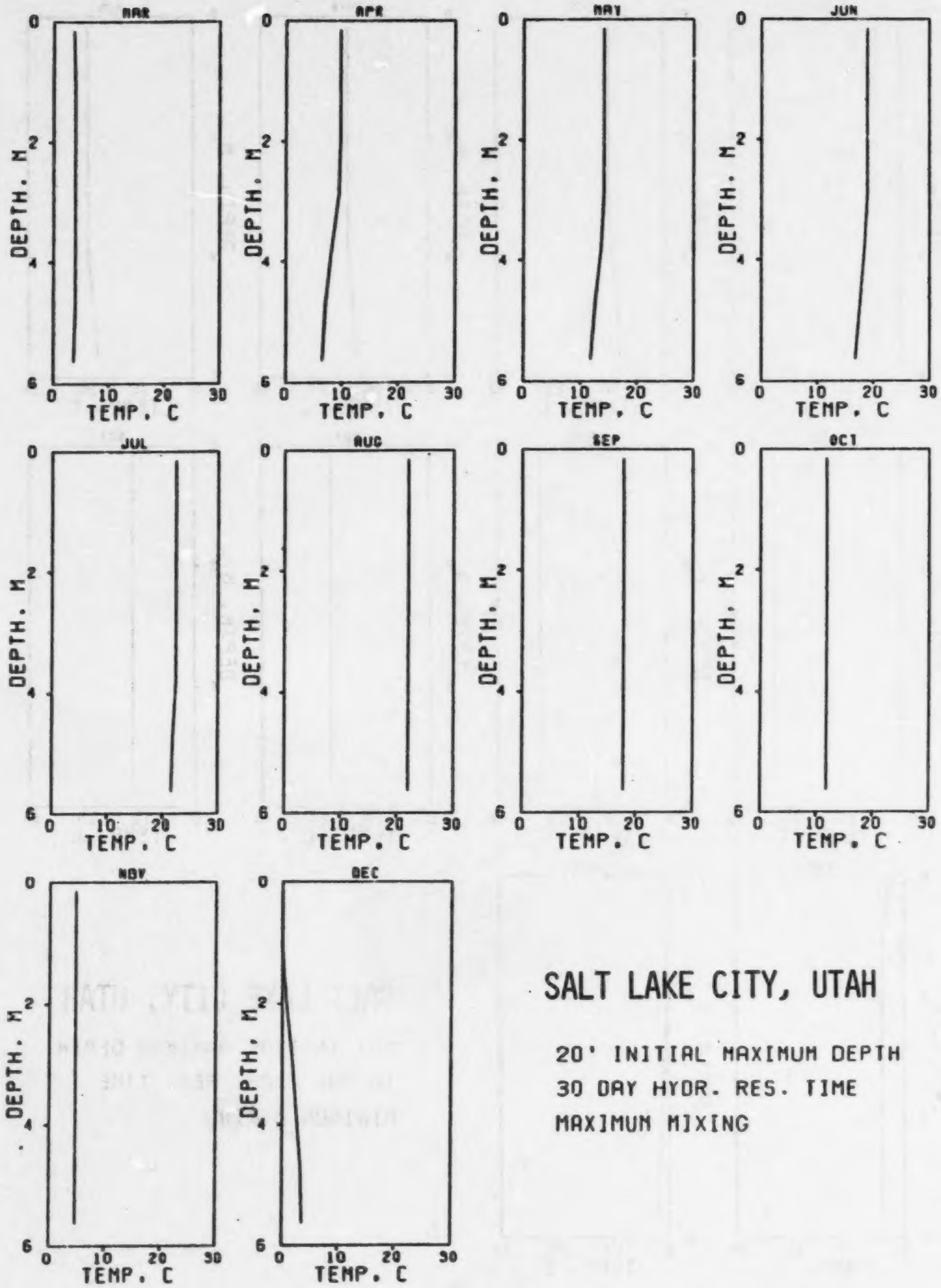
SALT LAKE CITY, UTAH

20' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



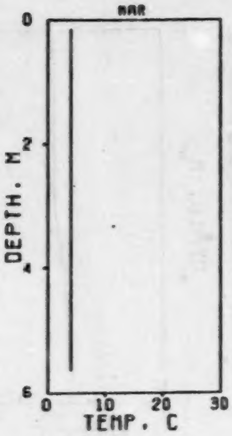
SALT LAKE CITY, UTAH

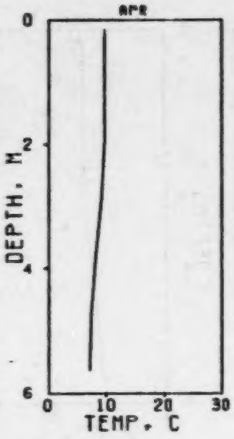
20' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING

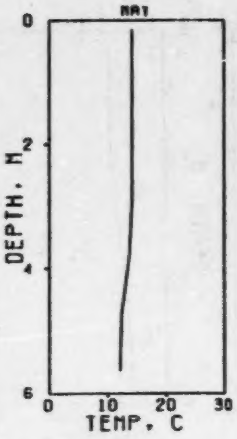


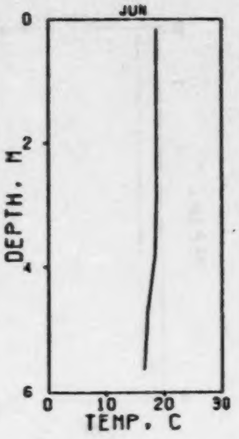
SALT LAKE CITY, UTAH

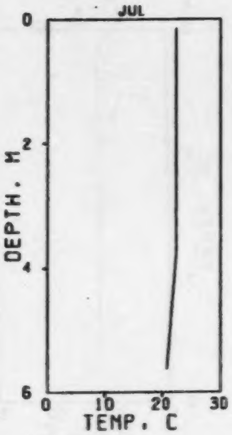
20' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING

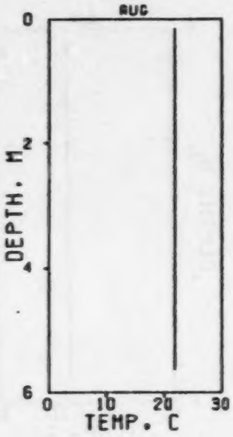


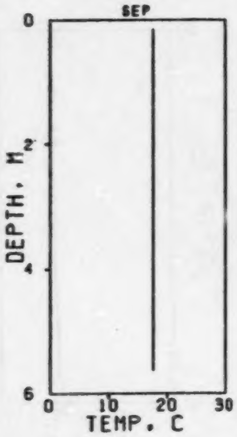


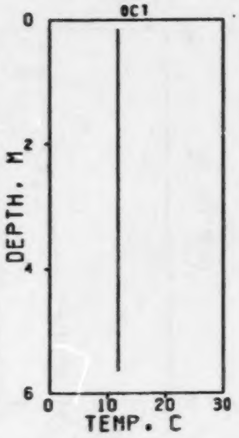


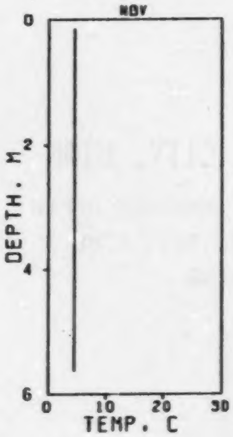


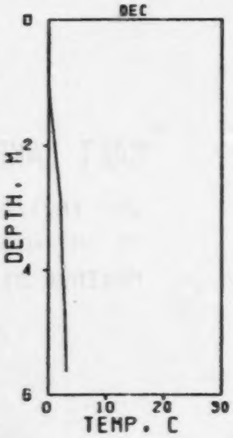






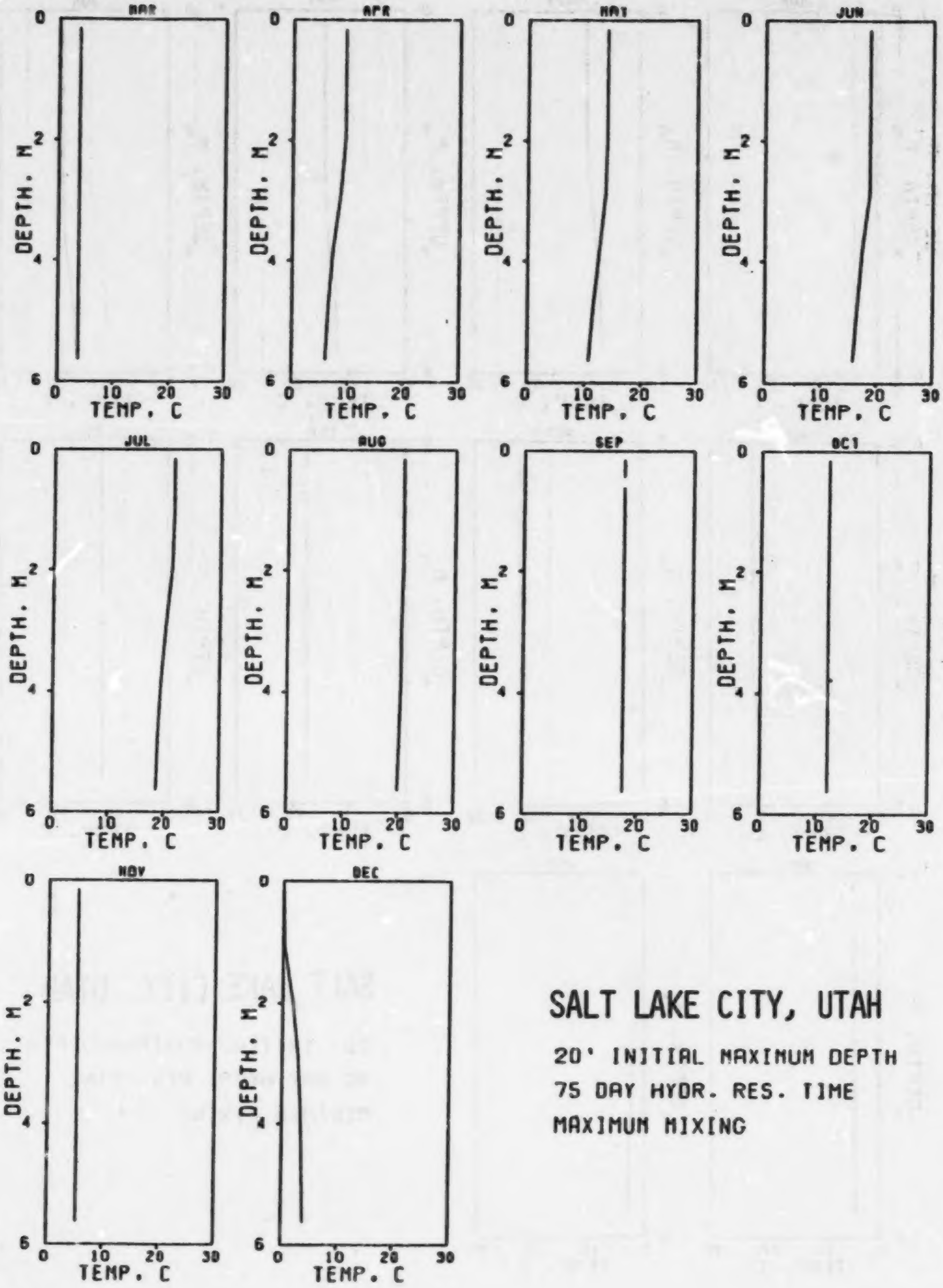






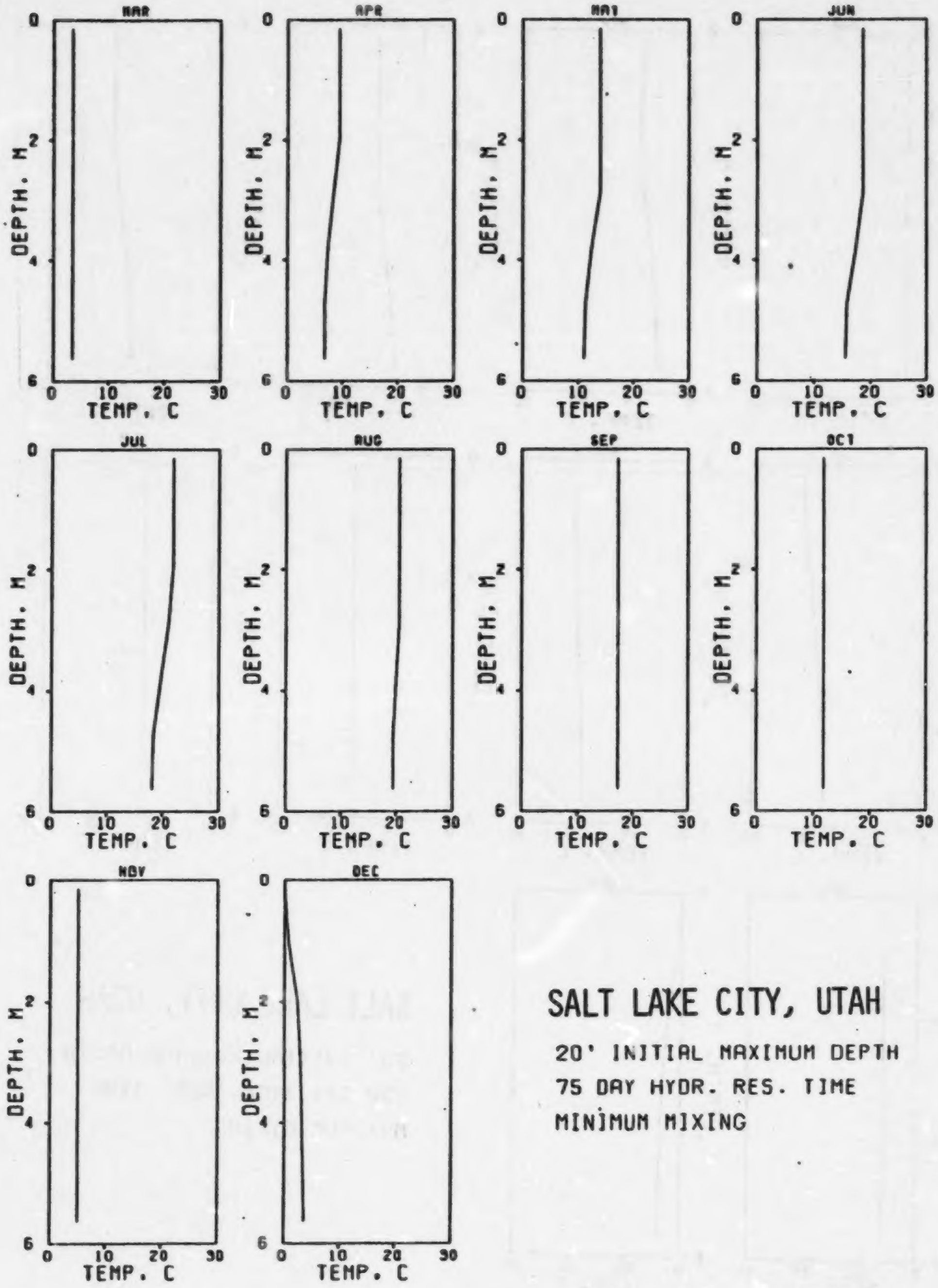
SALT LAKE CITY, UTAH

20' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



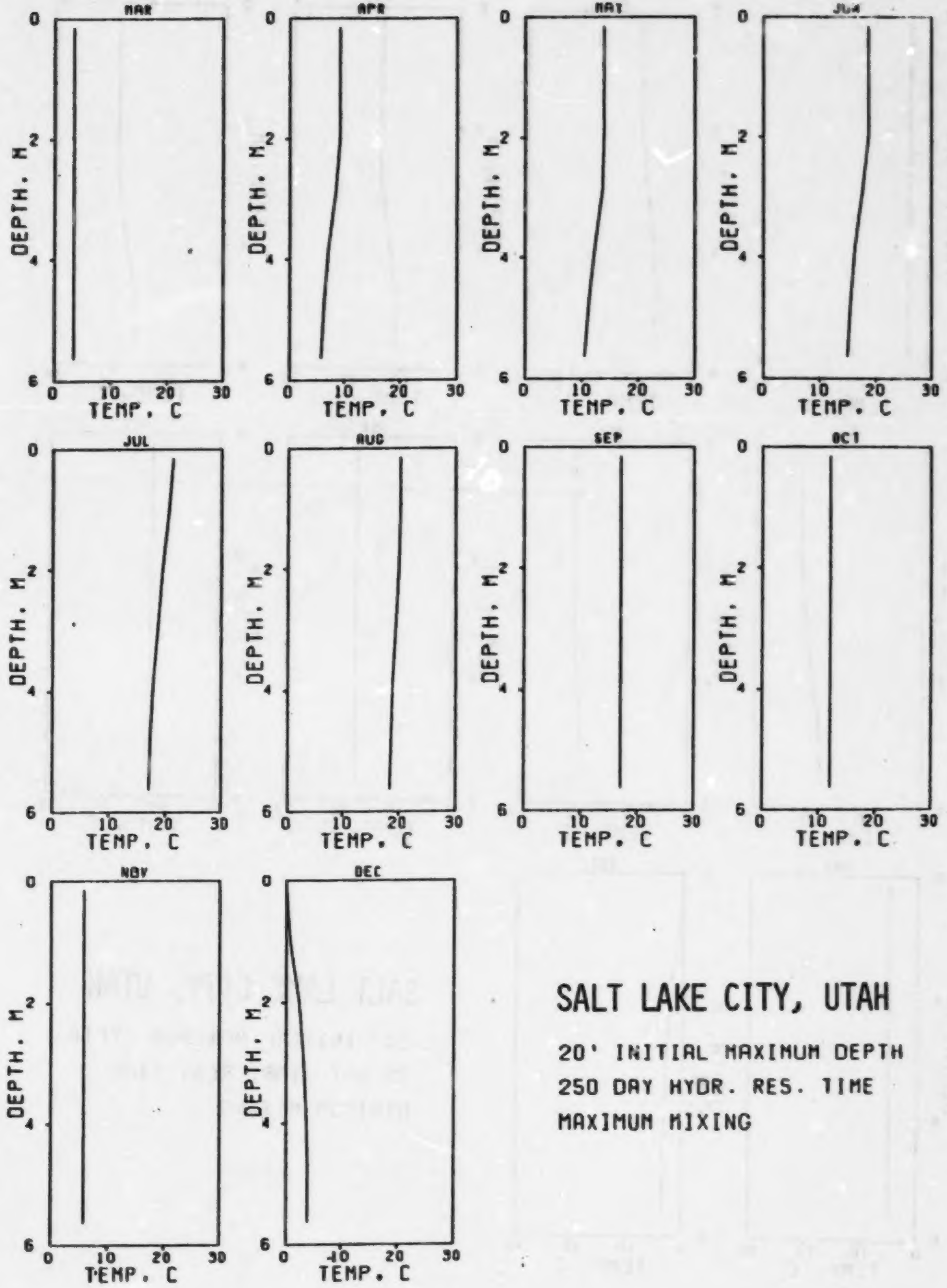
SALT LAKE CITY, UTAH

20' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



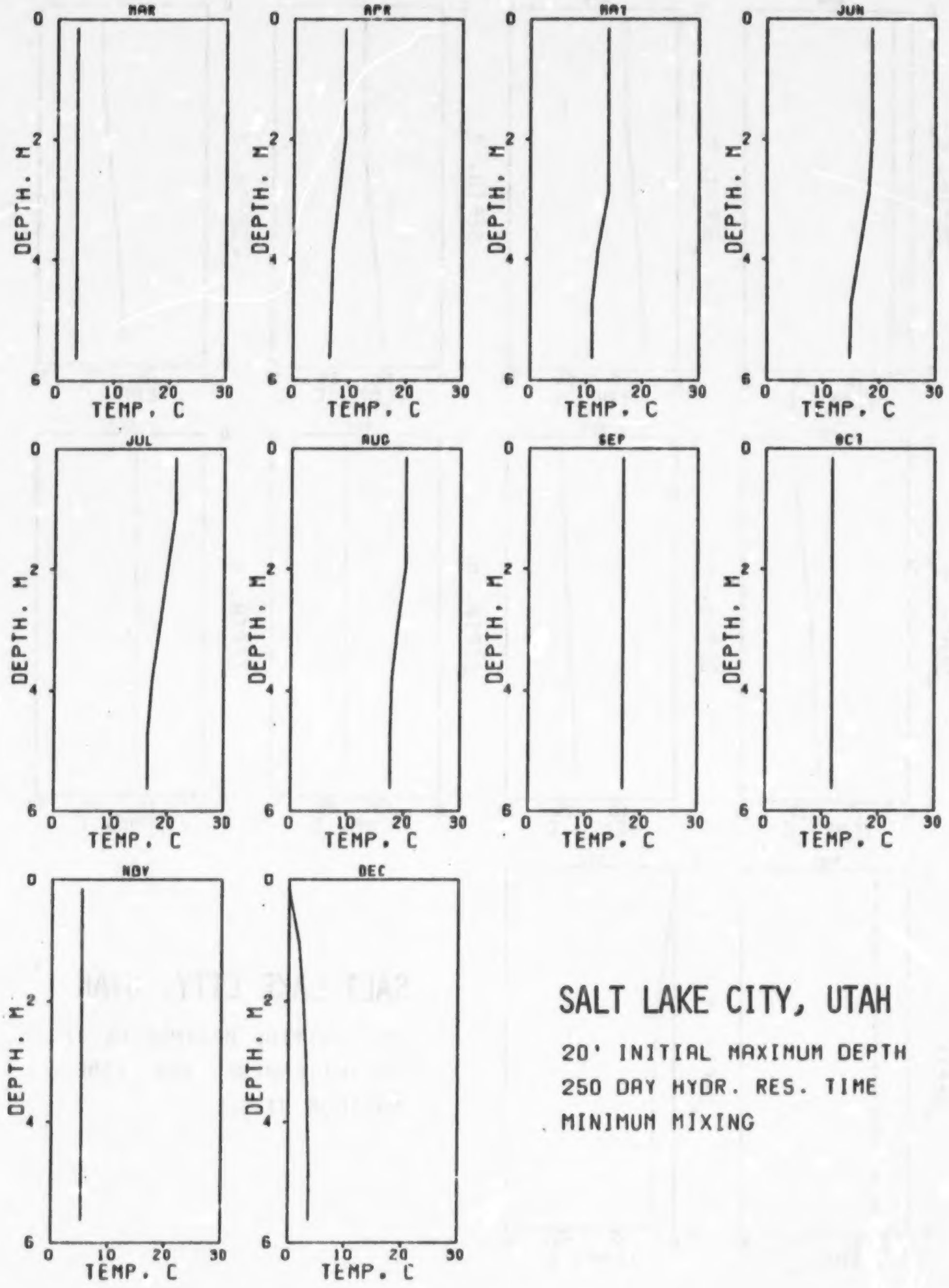
SALT LAKE CITY, UTAH

20' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



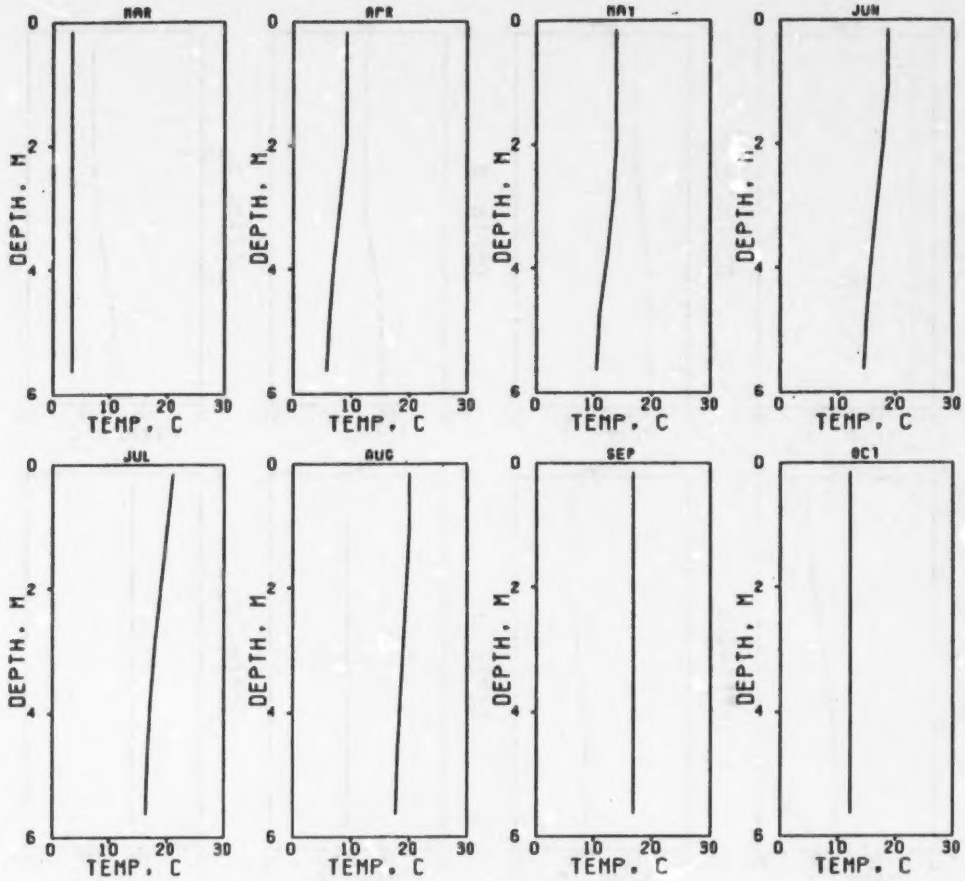
SALT LAKE CITY, UTAH

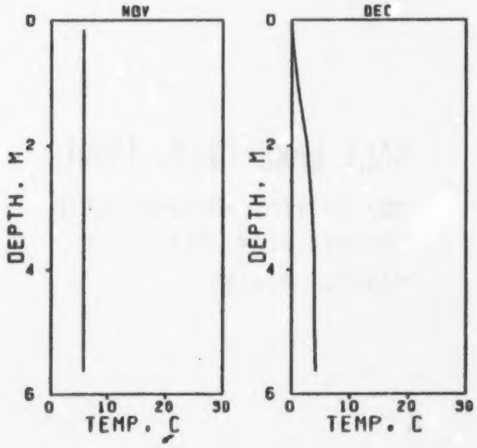
20' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



SALT LAKE CITY, UTAH

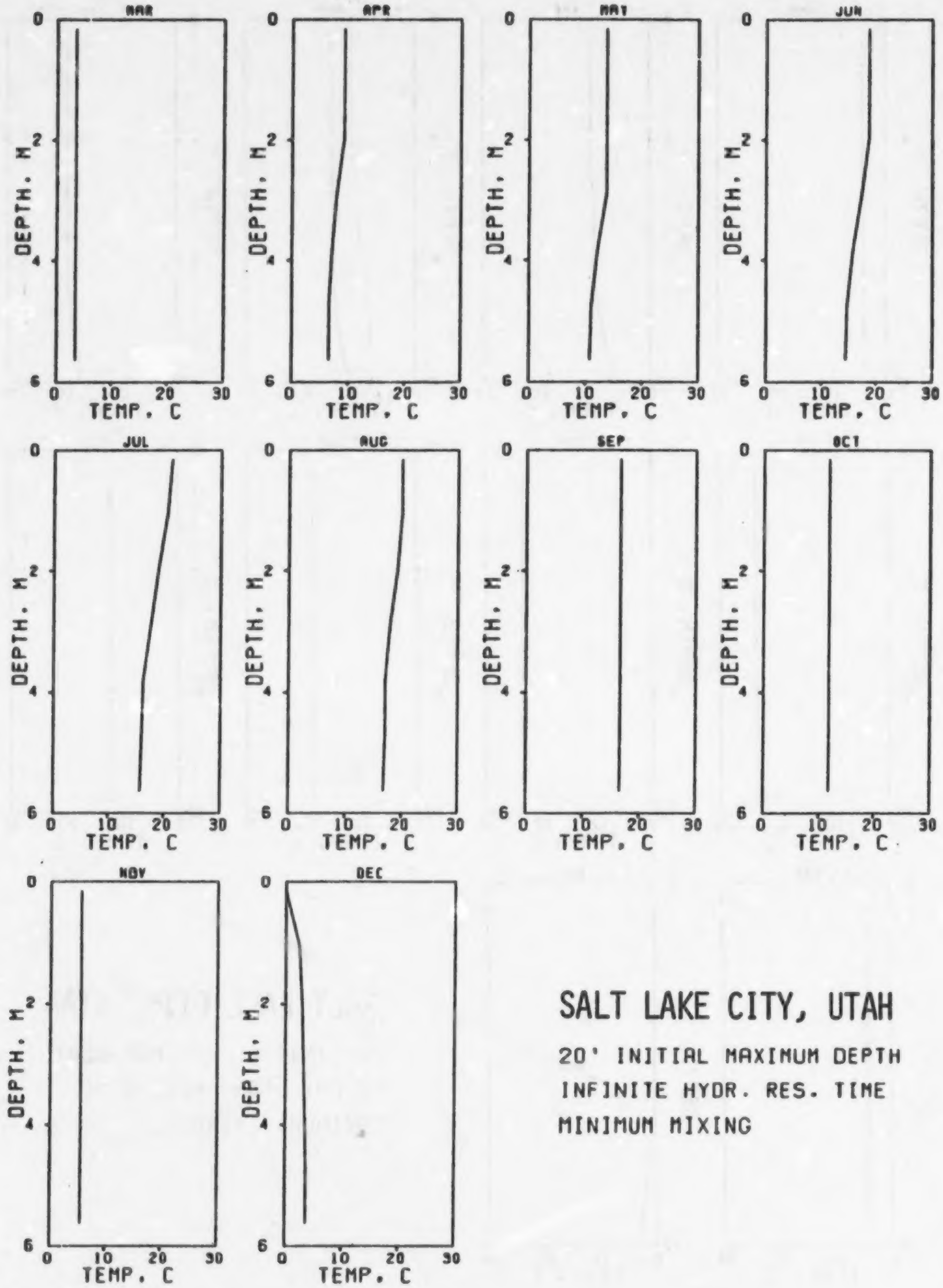
20' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING





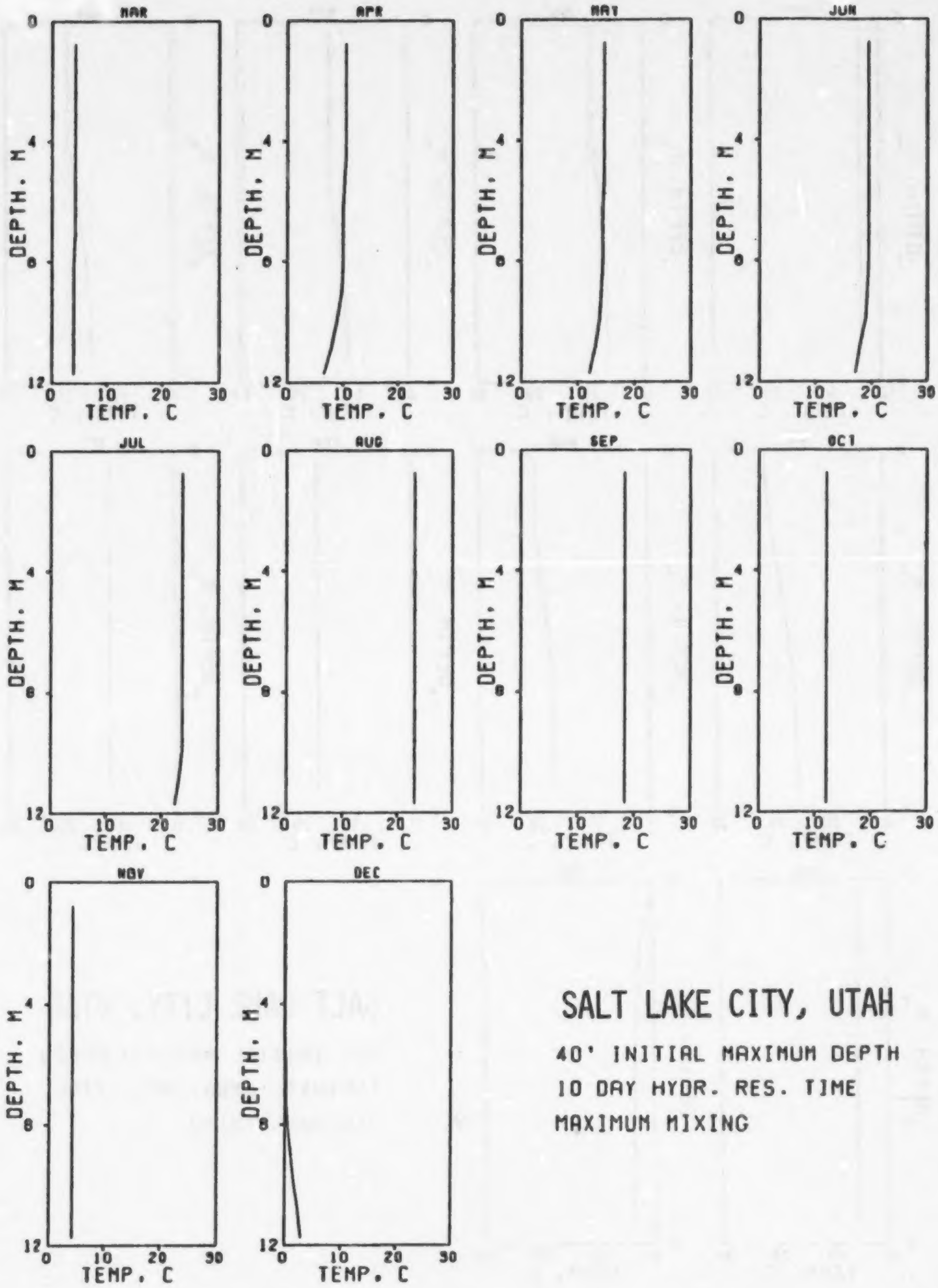
SALT LAKE CITY, UTAH

20' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



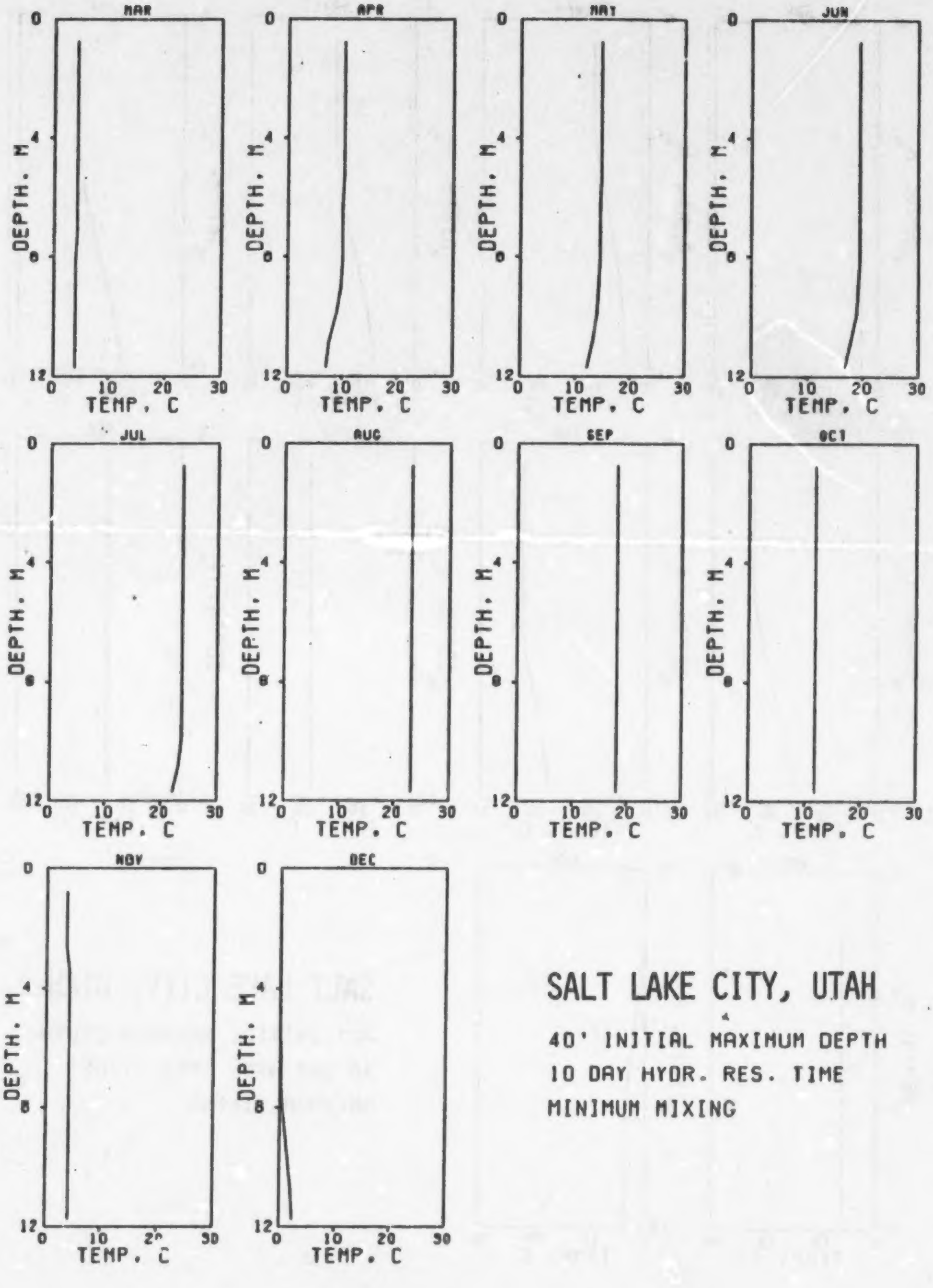
SALT LAKE CITY, UTAH

20' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



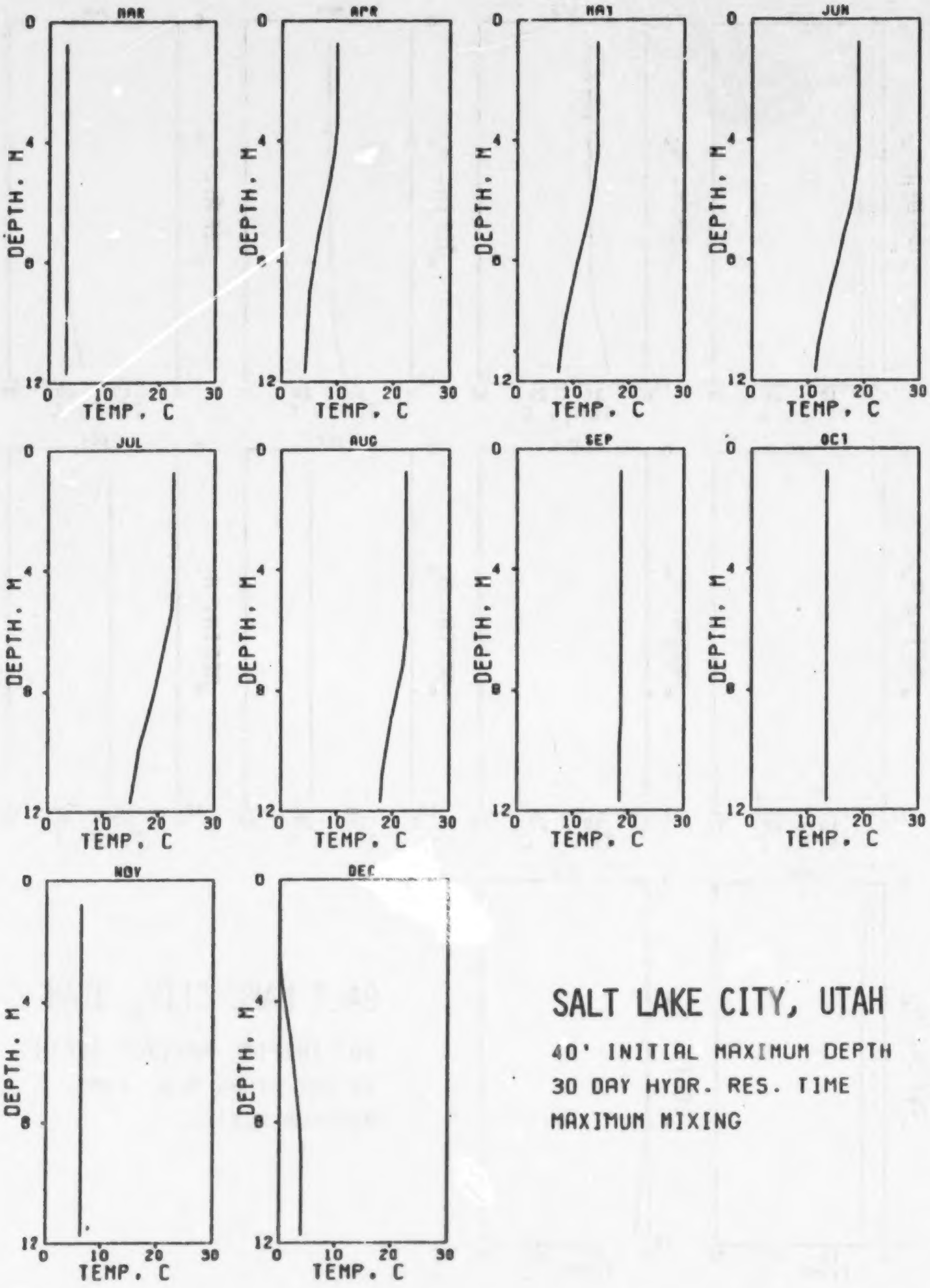
SALT LAKE CITY, UTAH

40' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



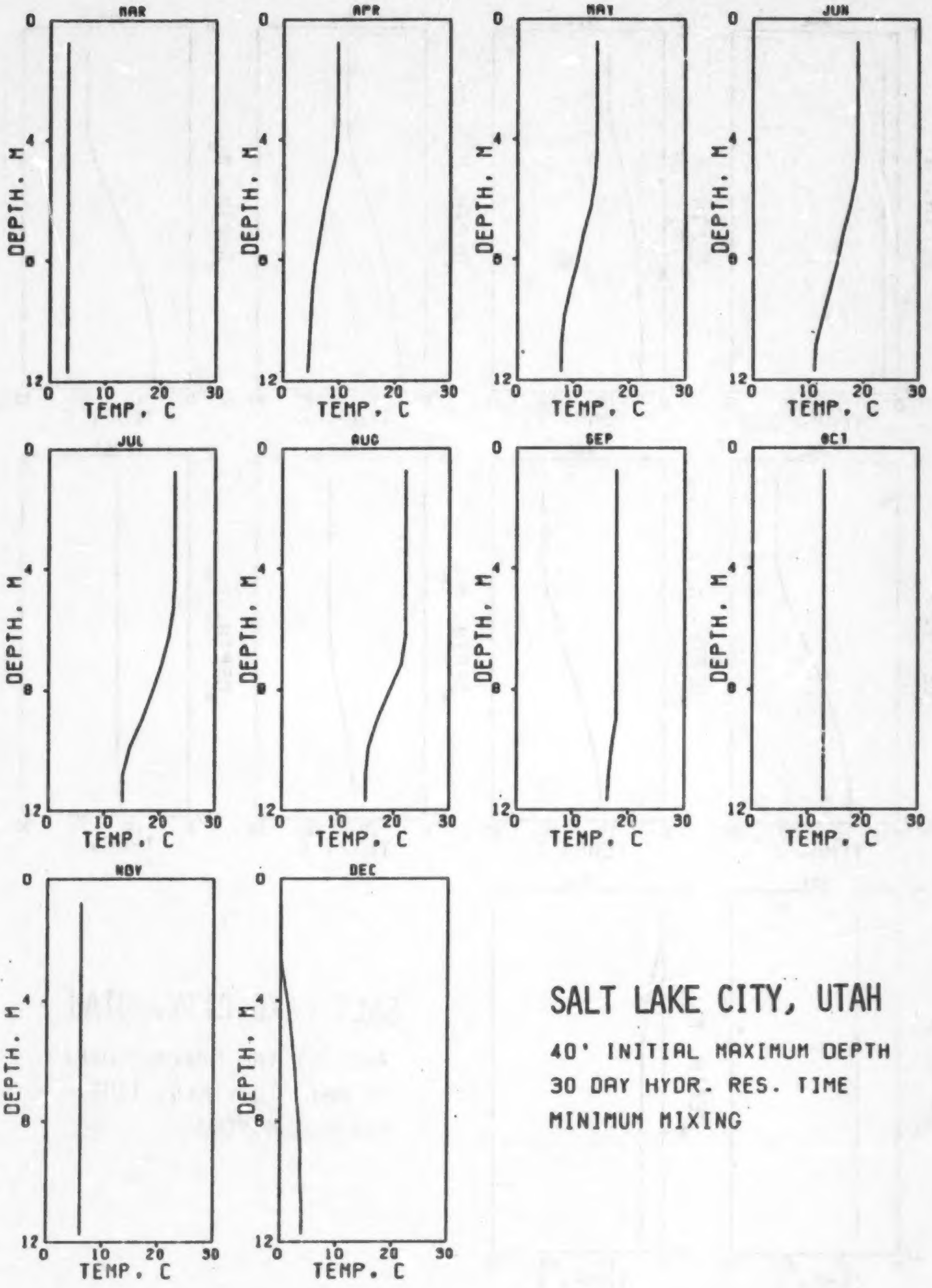
SALT LAKE CITY, UTAH

40' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



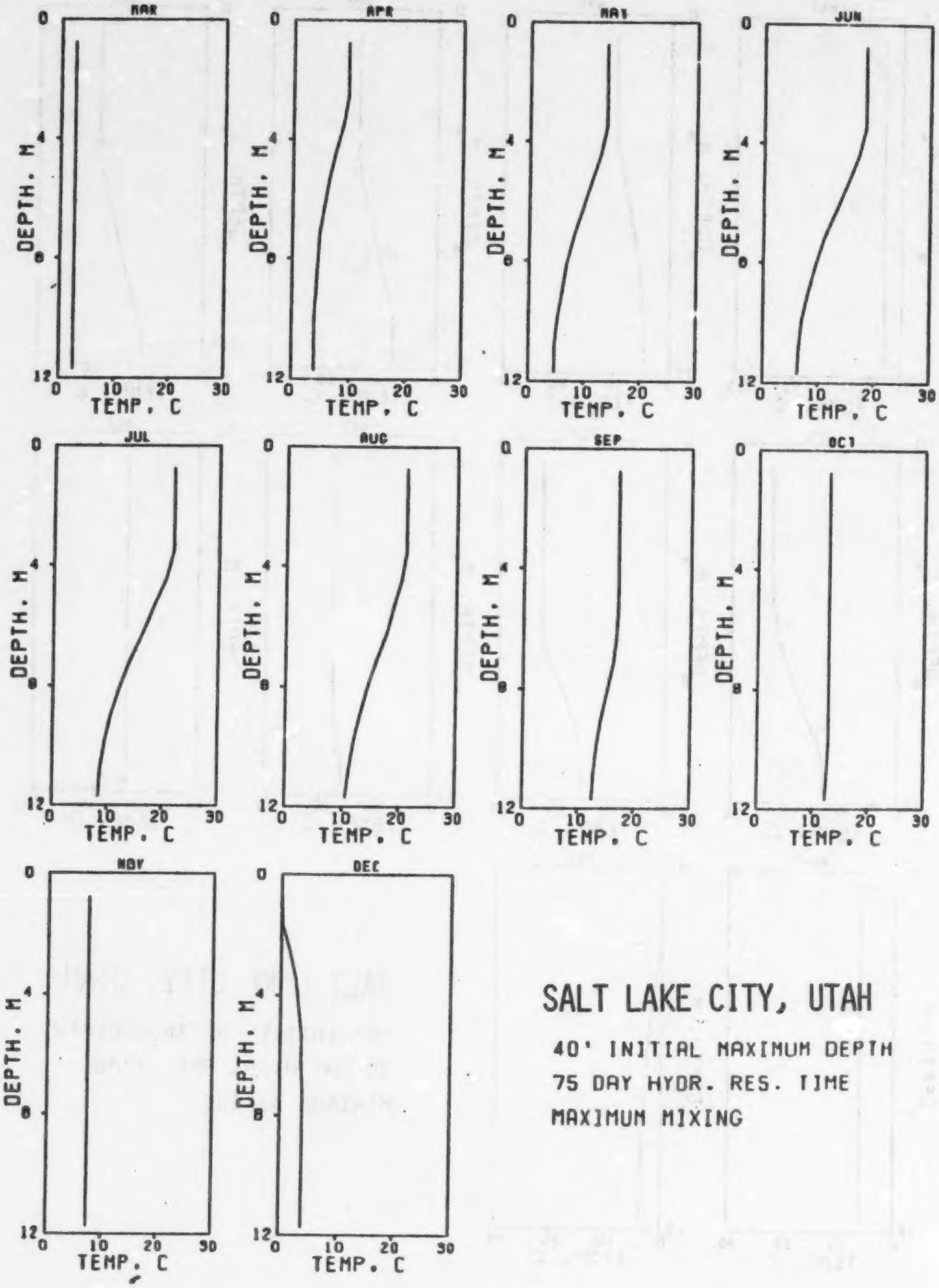
SALT LAKE CITY, UTAH

40' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



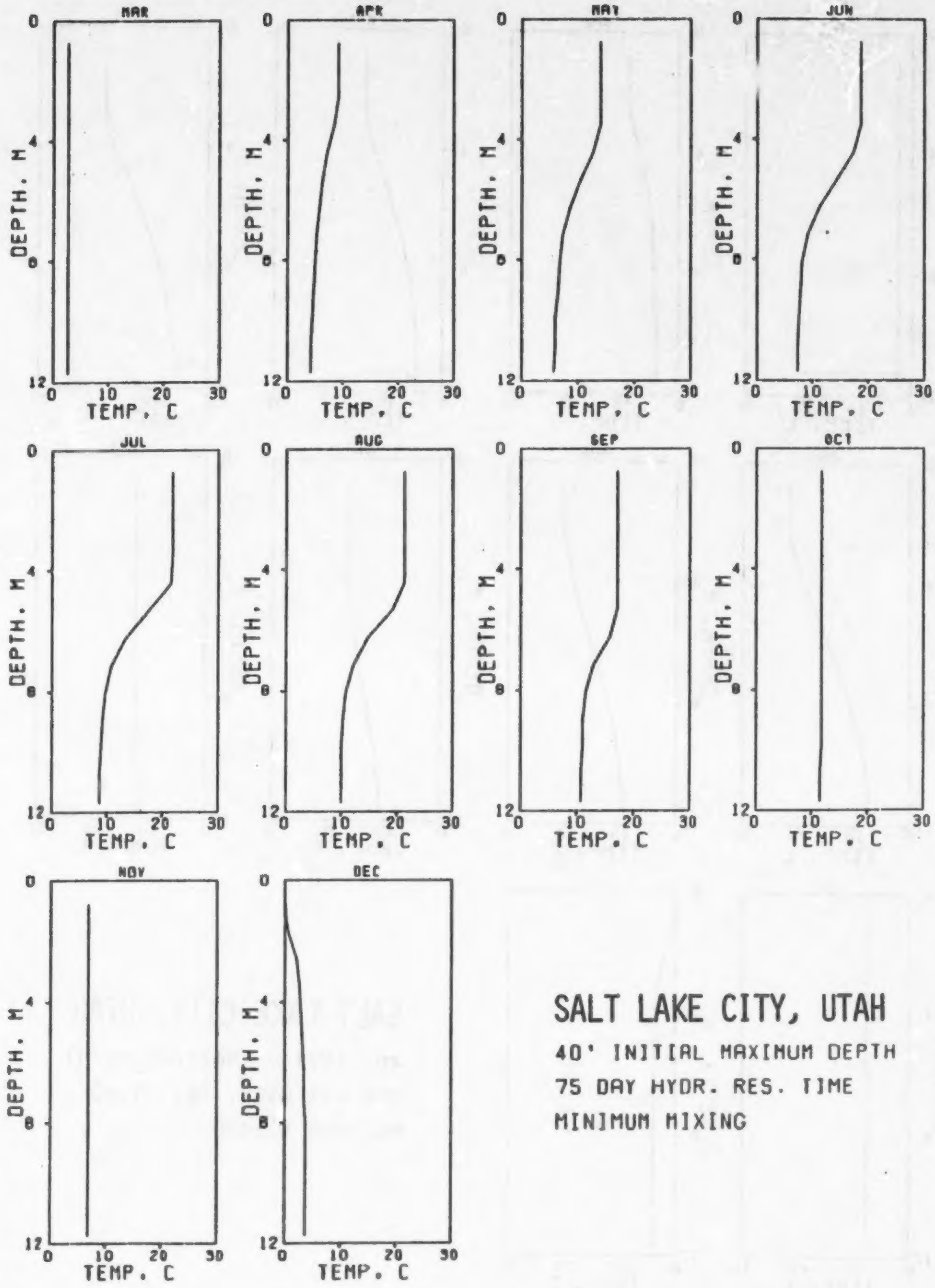
SALT LAKE CITY, UTAH

40' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



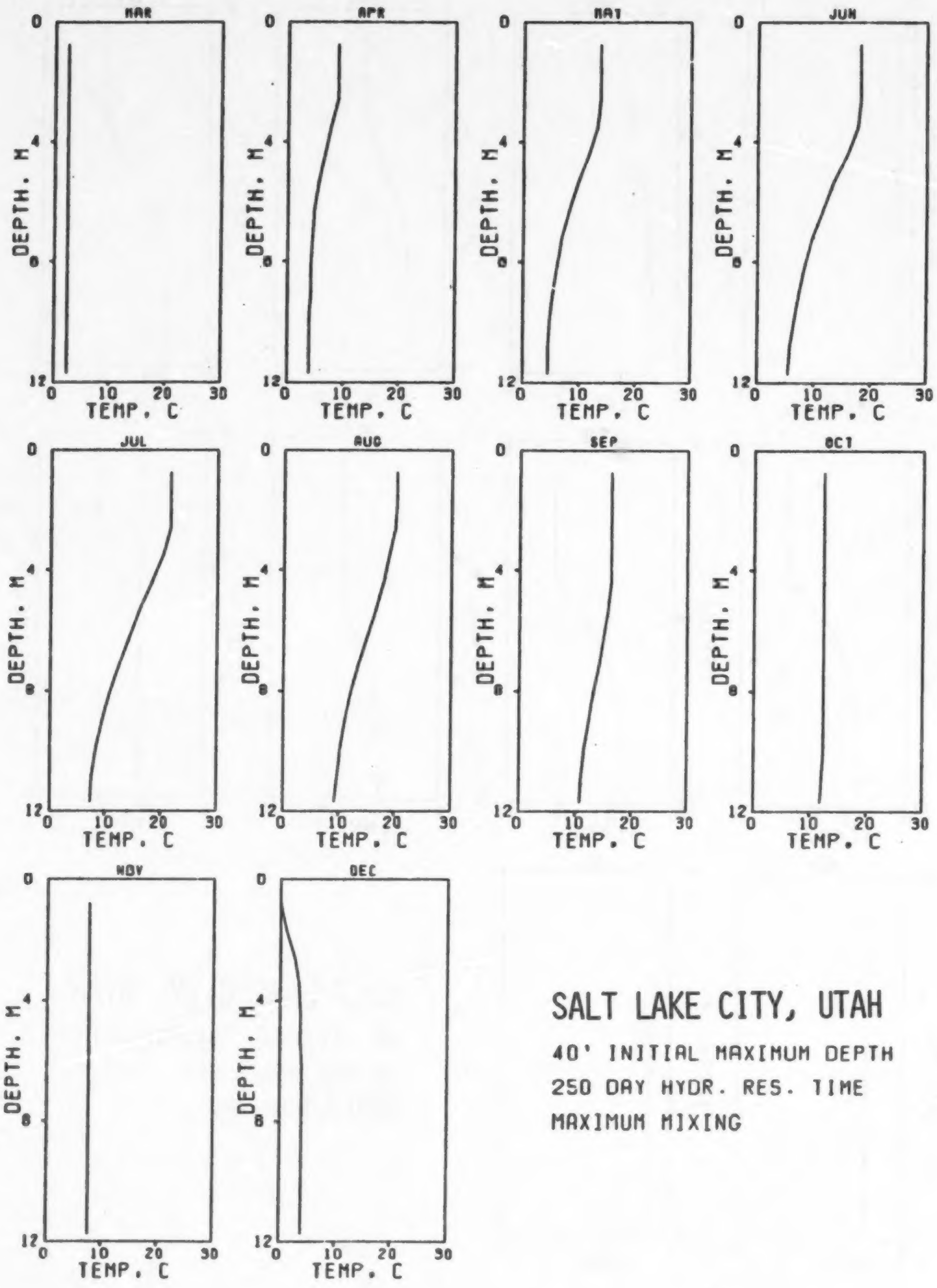
SALT LAKE CITY, UTAH

40' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



SALT LAKE CITY, UTAH

40' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING

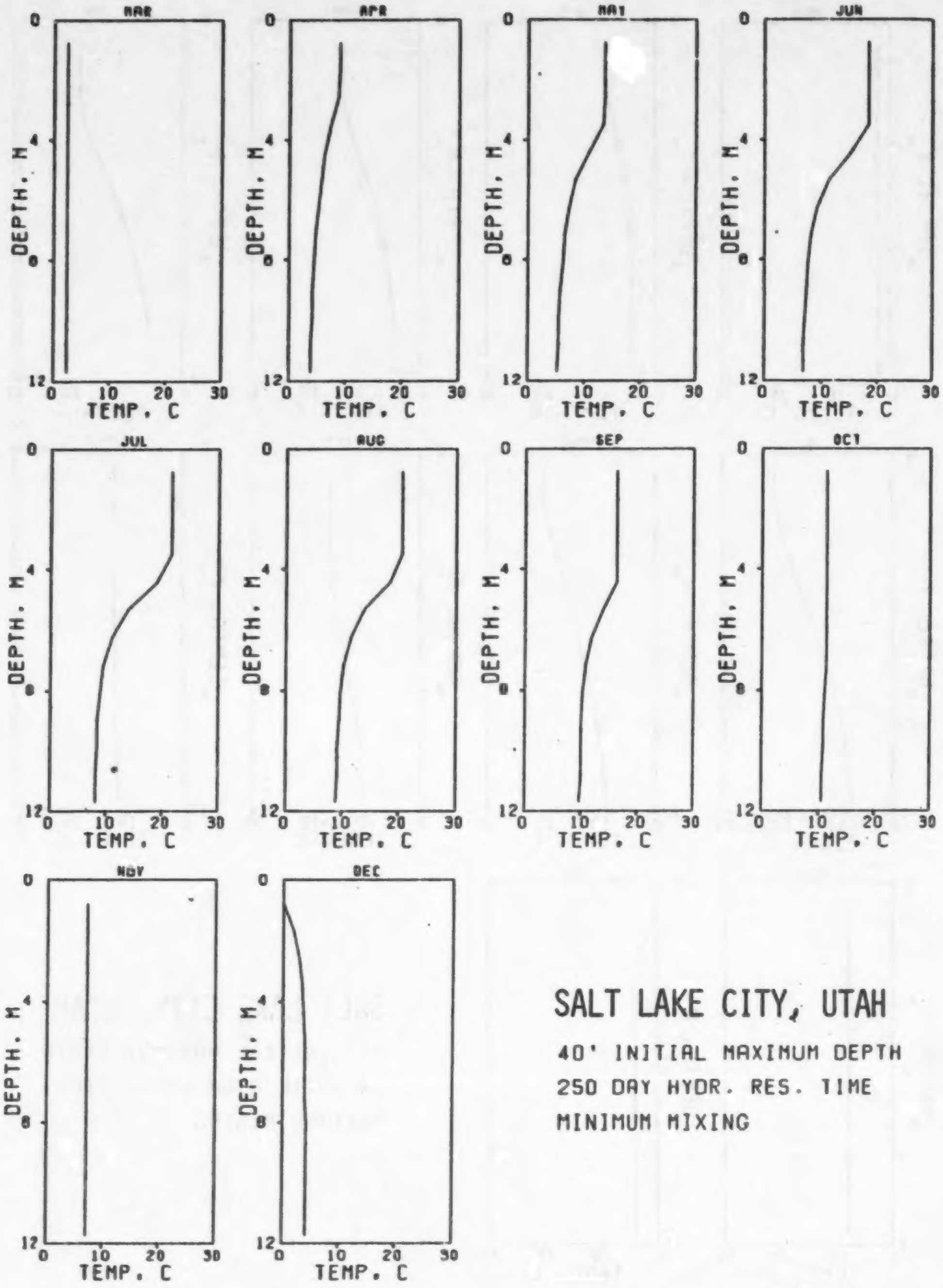


SALT LAKE CITY, UTAH

40' INITIAL MAXIMUM DEPTH

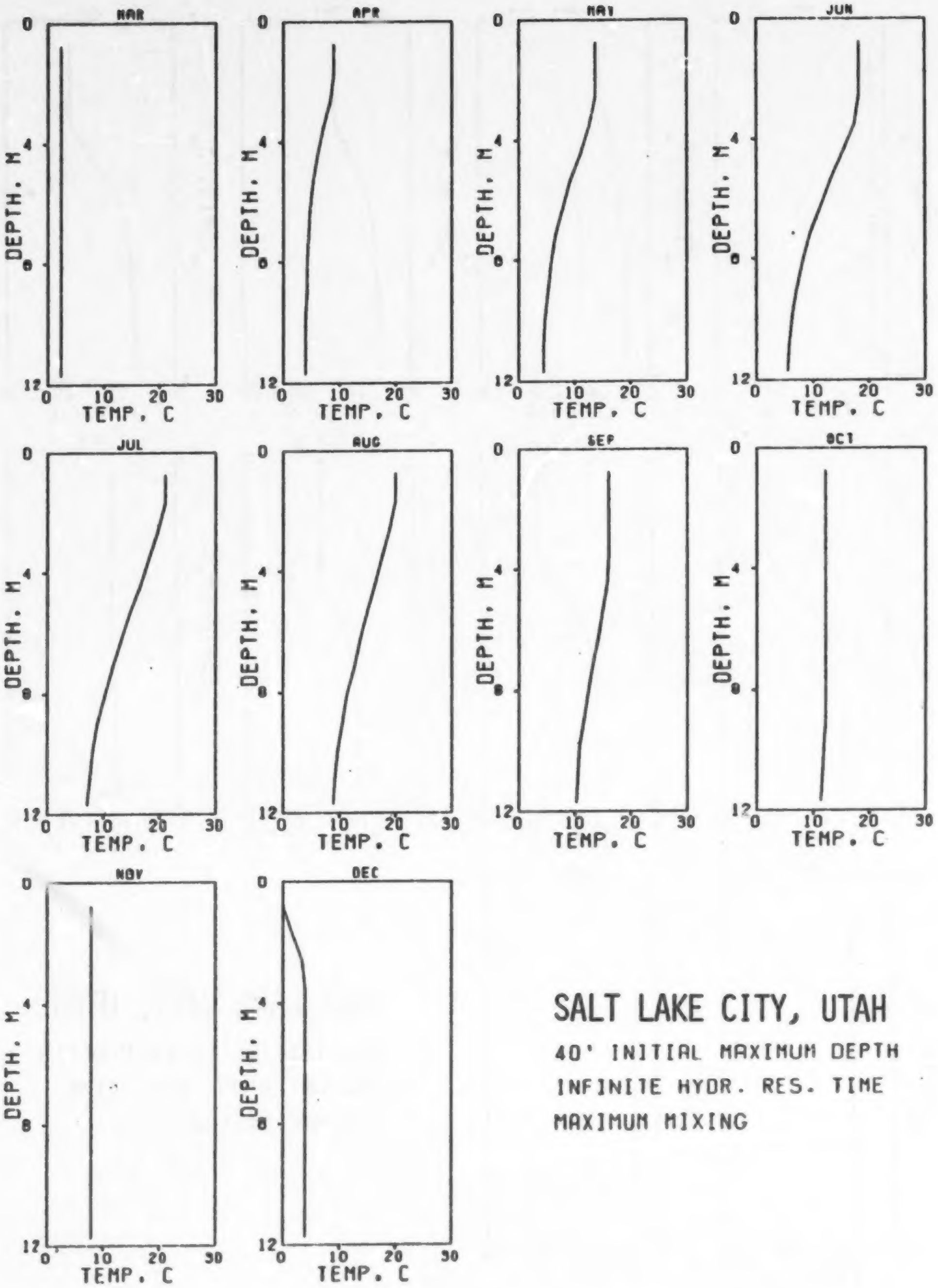
250 DAY HYDR. RES. TIME

MAXIMUM MIXING



SALT LAKE CITY, UTAH

40' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING

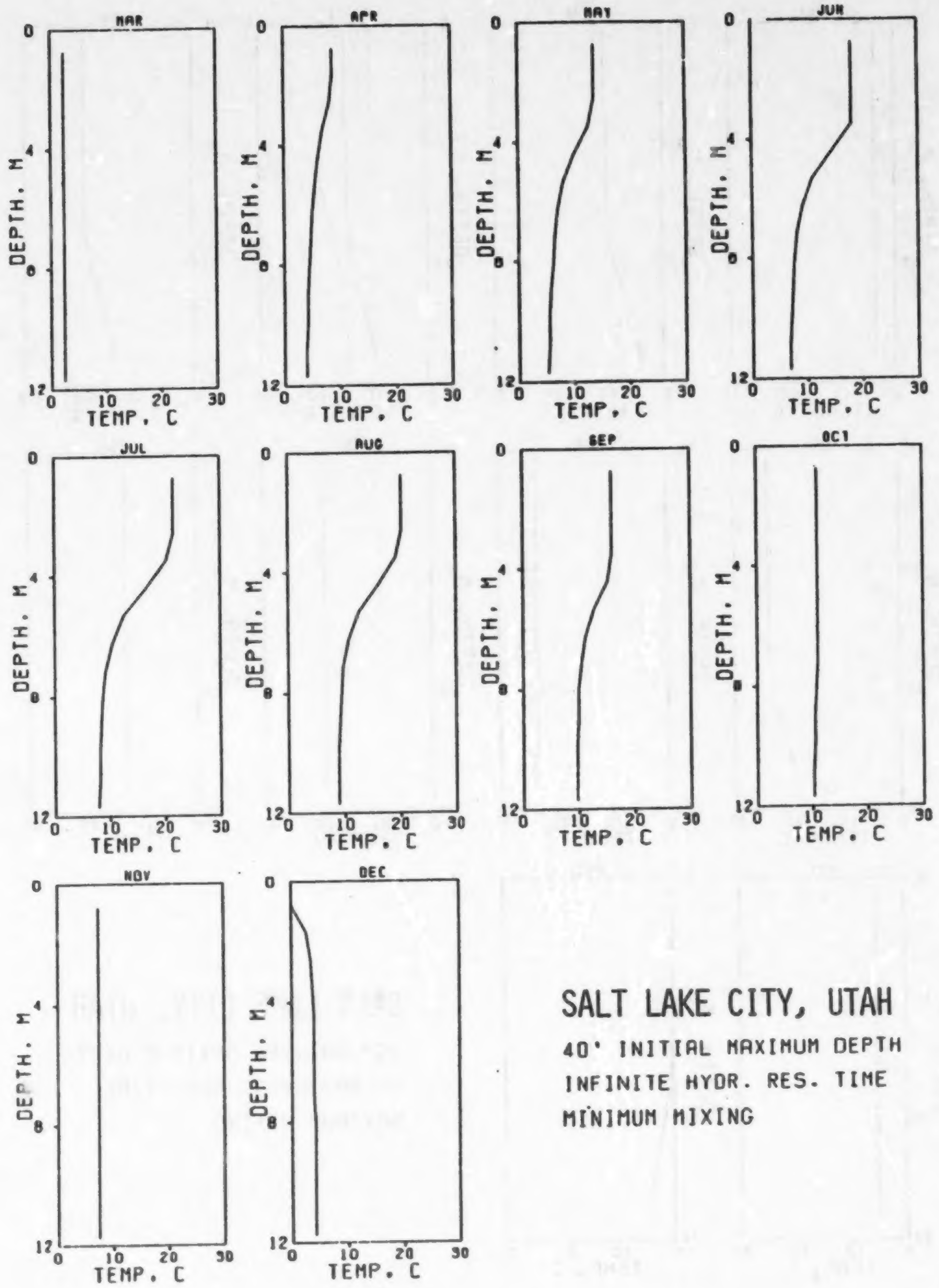


SALT LAKE CITY, UTAH

40' INITIAL MAXIMUM DEPTH

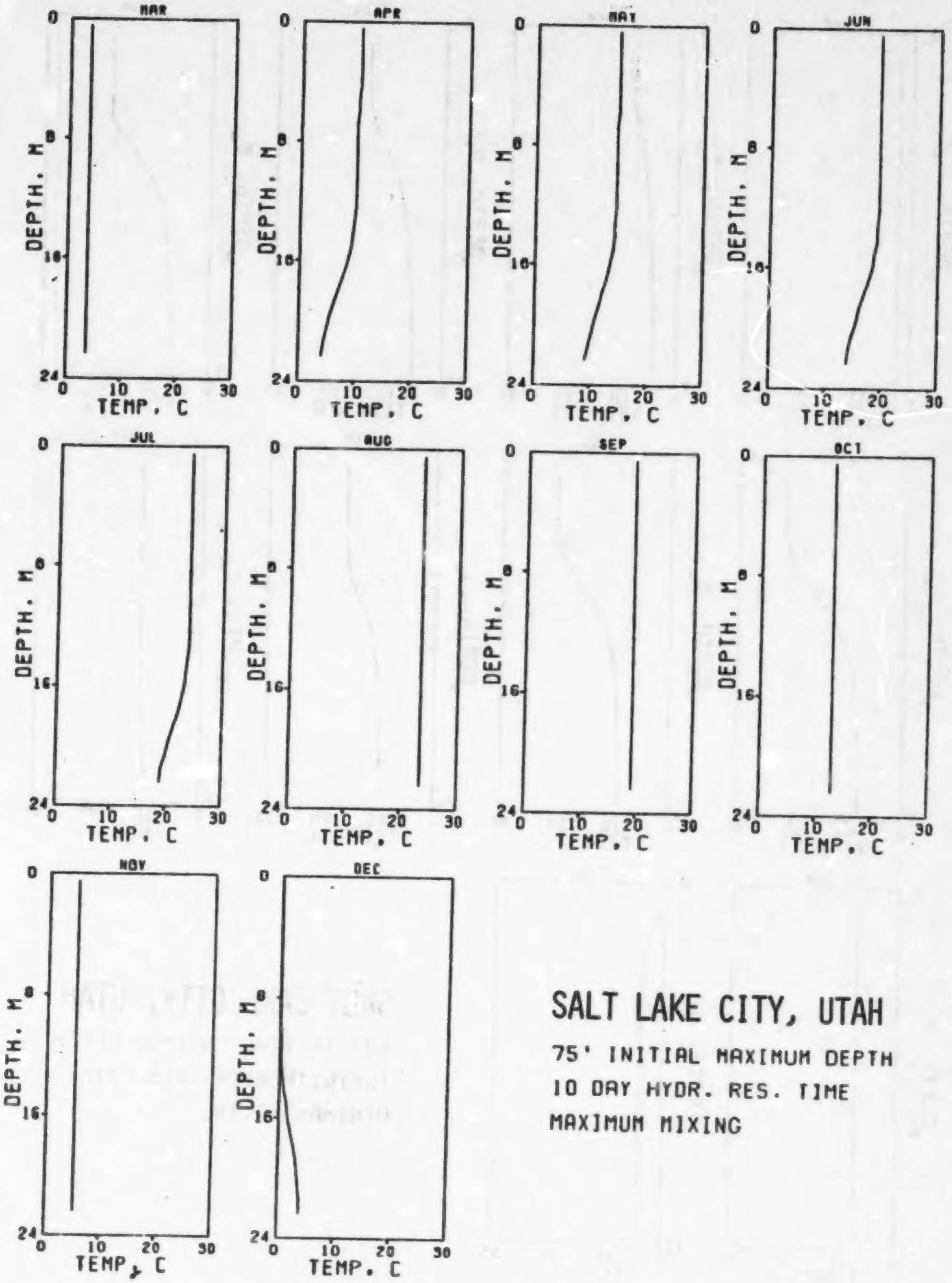
INFINITE HYDR. RES. TIME

MAXIMUM MIXING



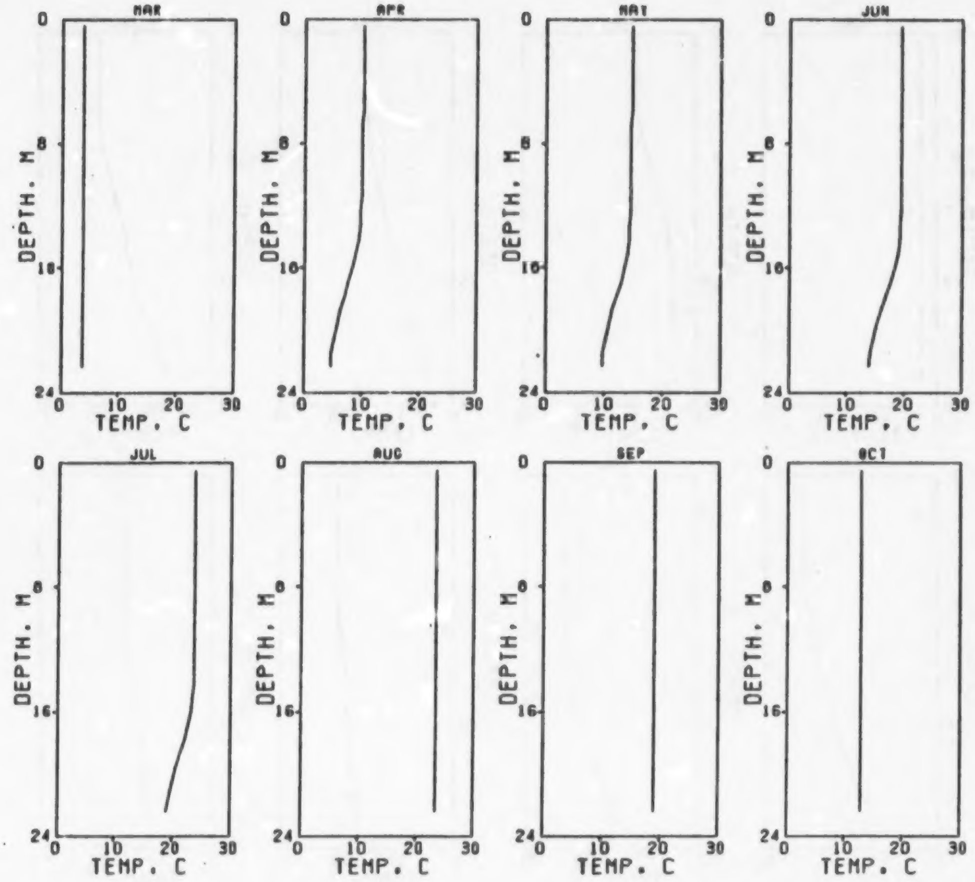
SALT LAKE CITY, UTAH

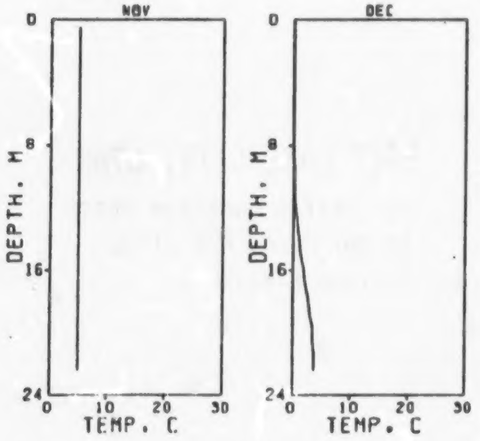
40' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



SALT LAKE CITY, UTAH

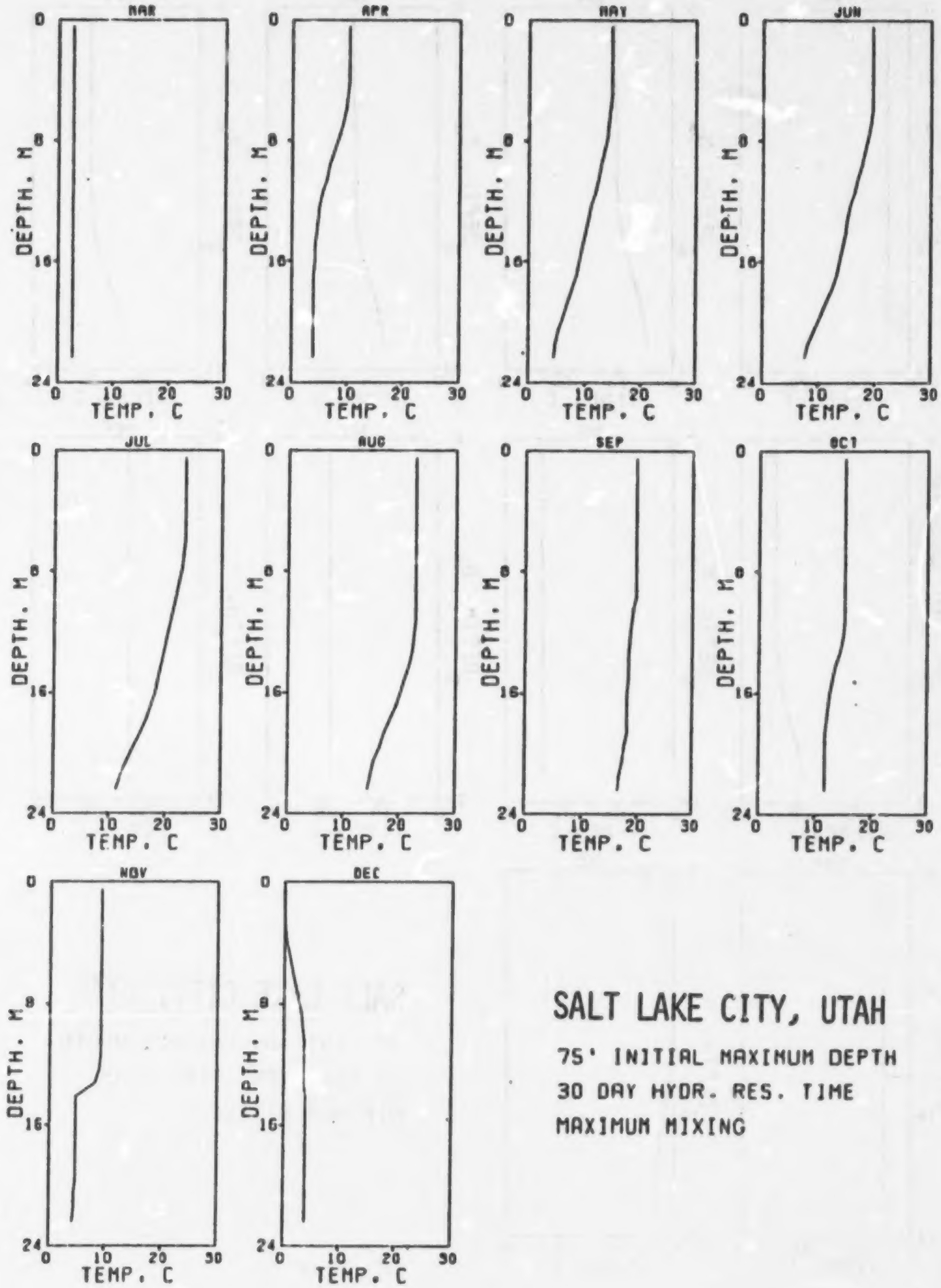
75' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING





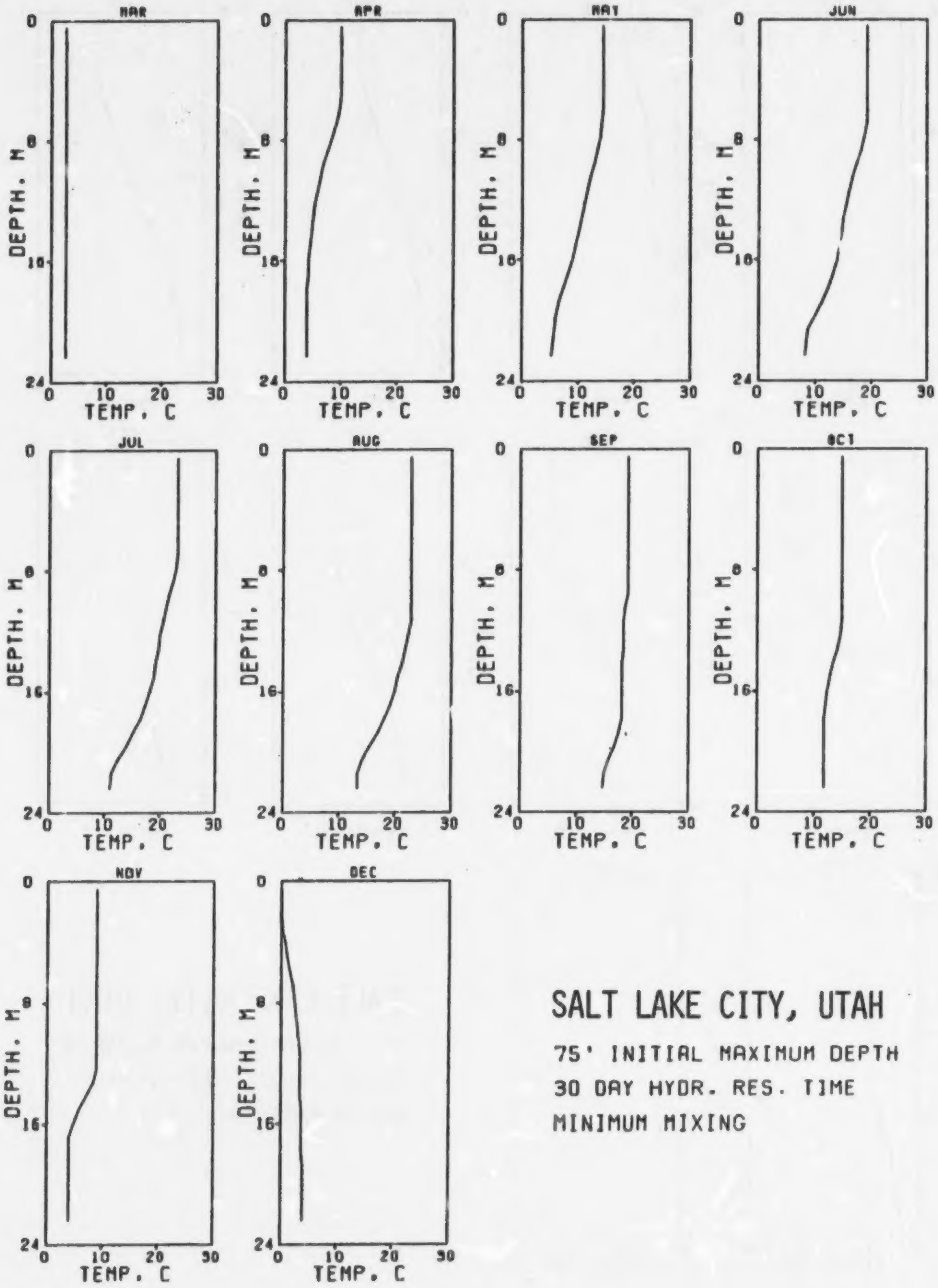
SALT LAKE CITY, UTAH

75' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



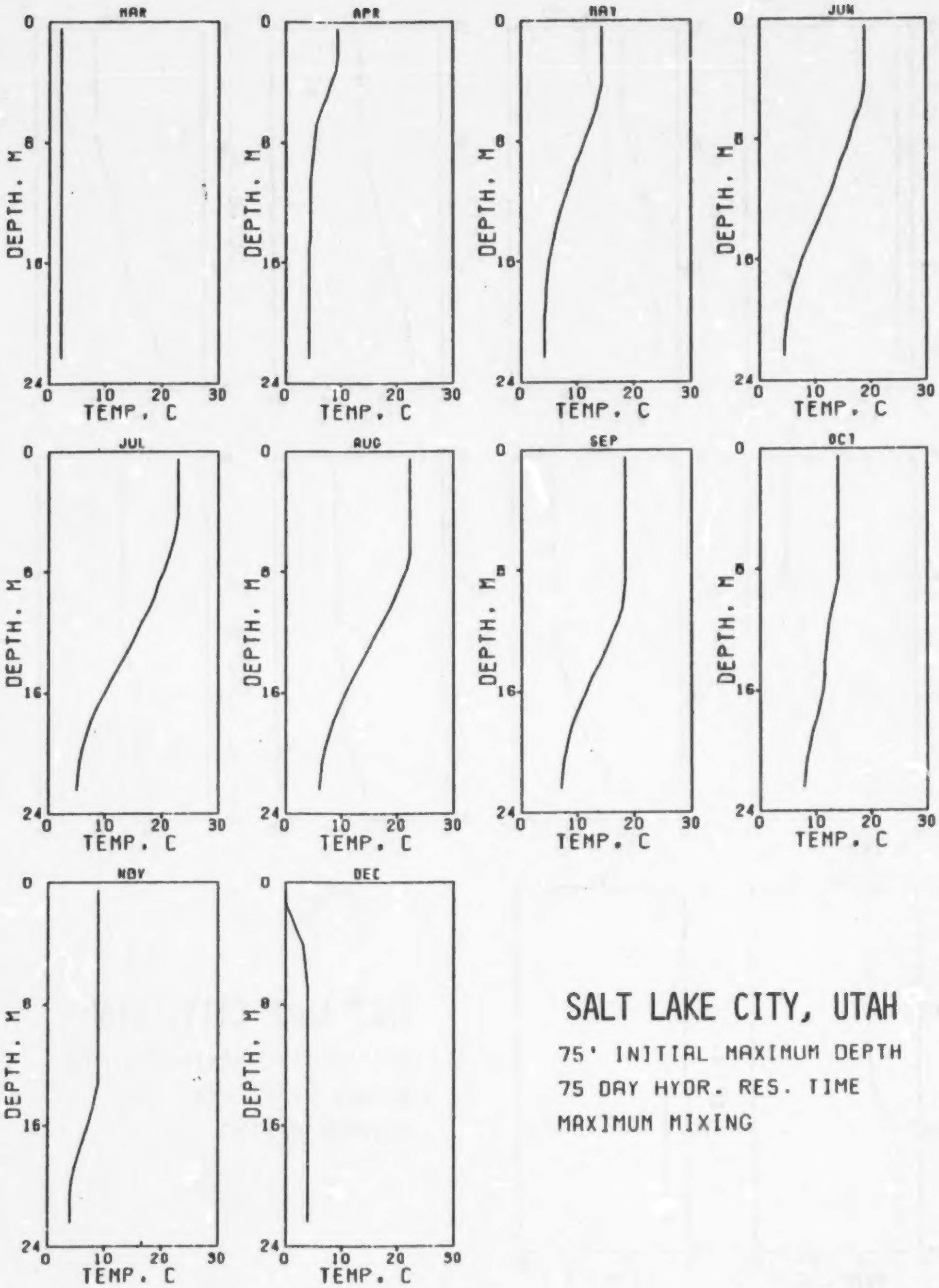
SALT LAKE CITY, UTAH

75' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



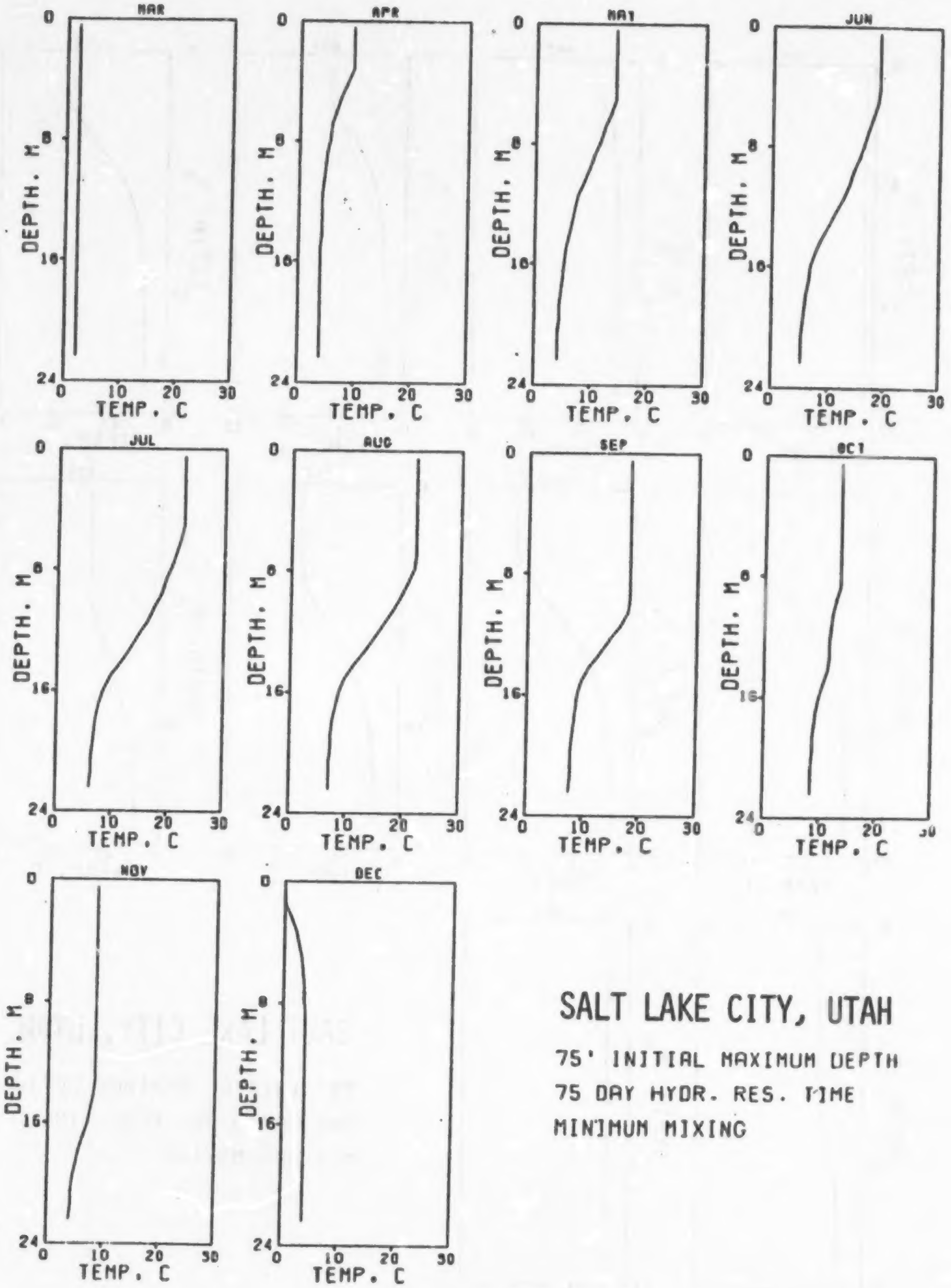
SALT LAKE CITY, UTAH

75' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



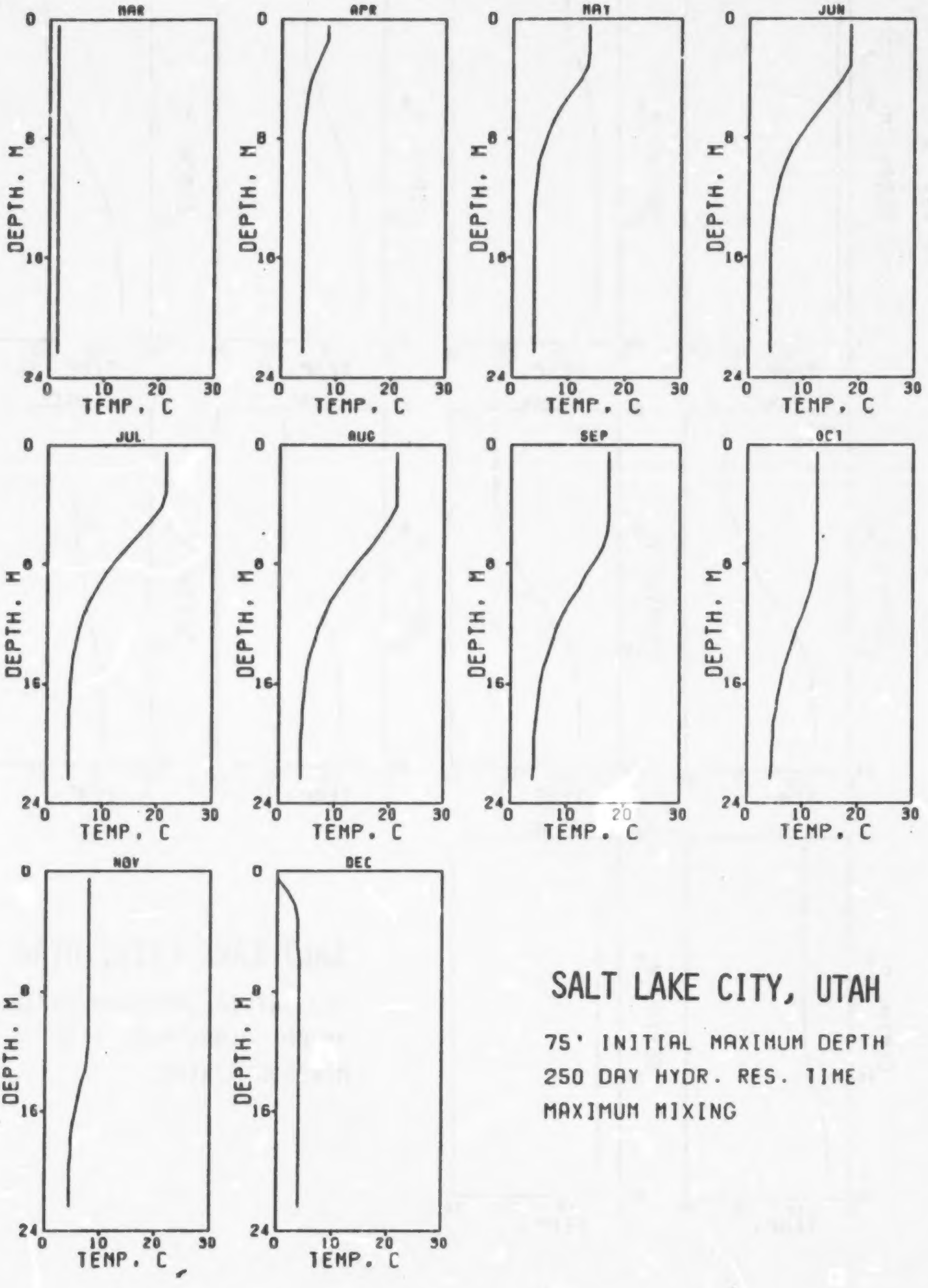
SALT LAKE CITY, UTAH

75' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



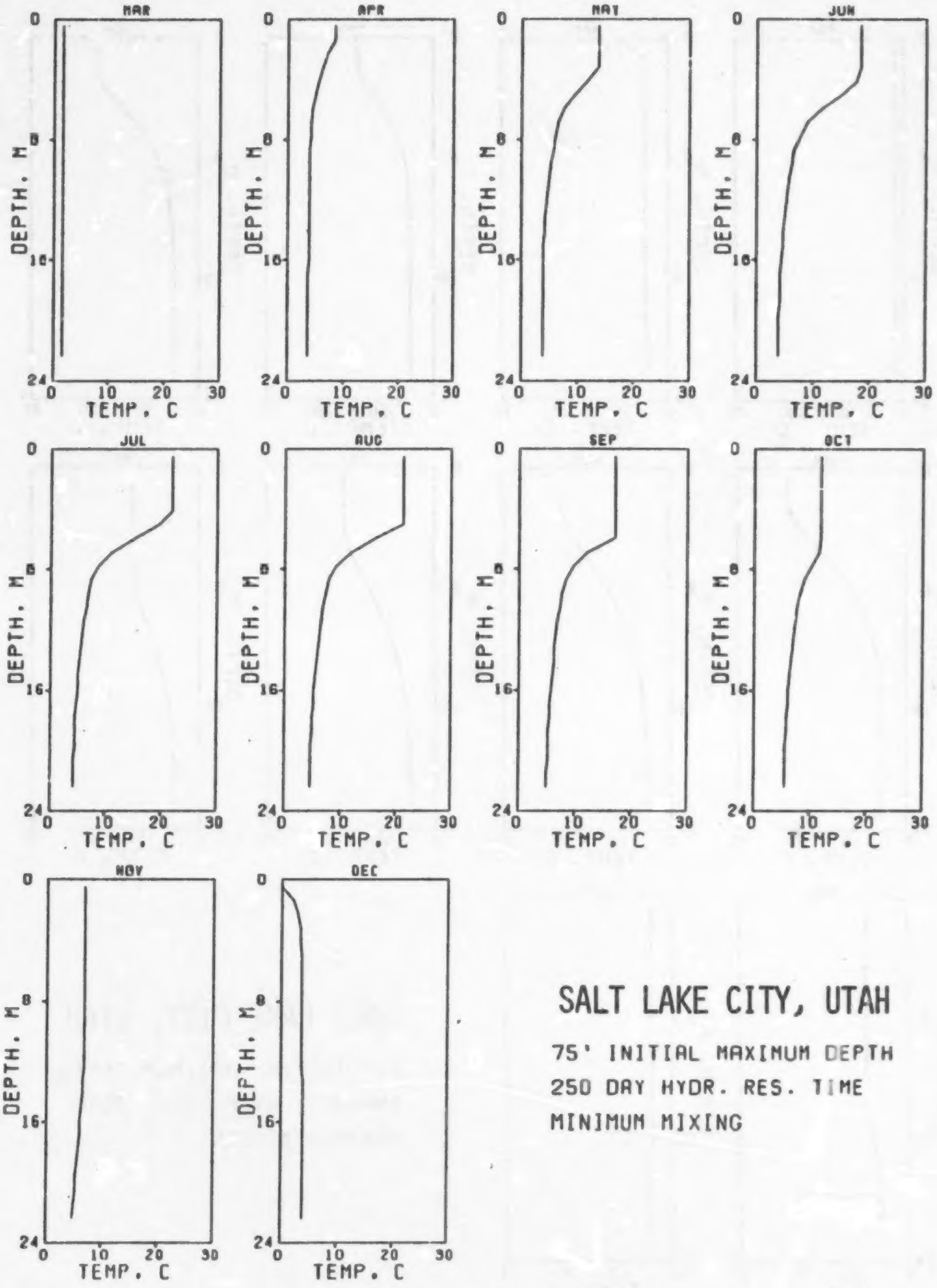
SALT LAKE CITY, UTAH

75' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



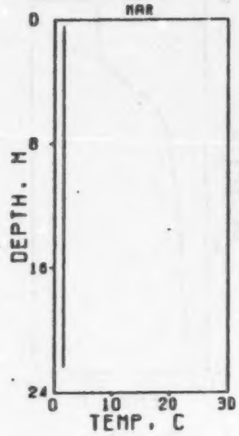
SALT LAKE CITY, UTAH

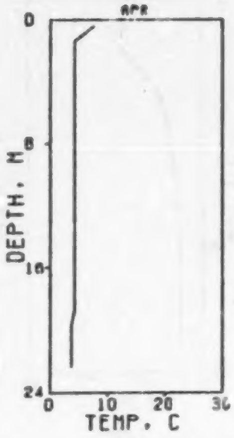
75' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING

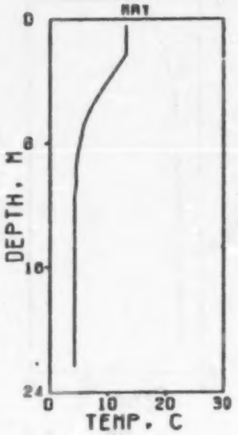


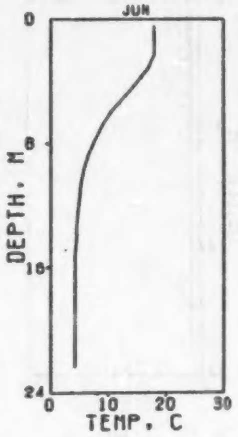
SALT LAKE CITY, UTAH

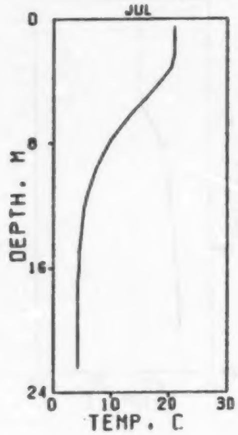
75' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING

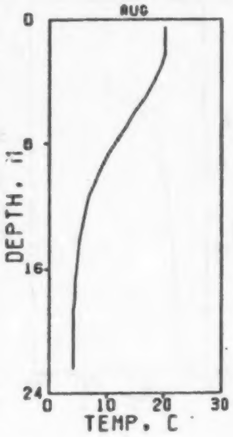


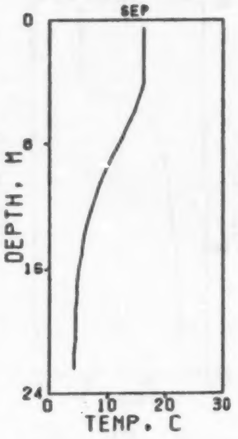


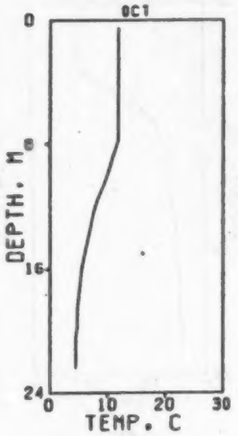


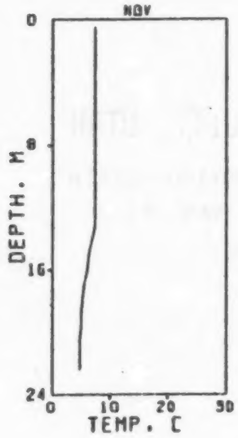


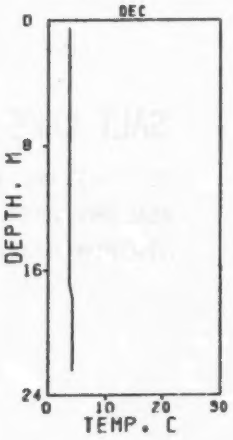






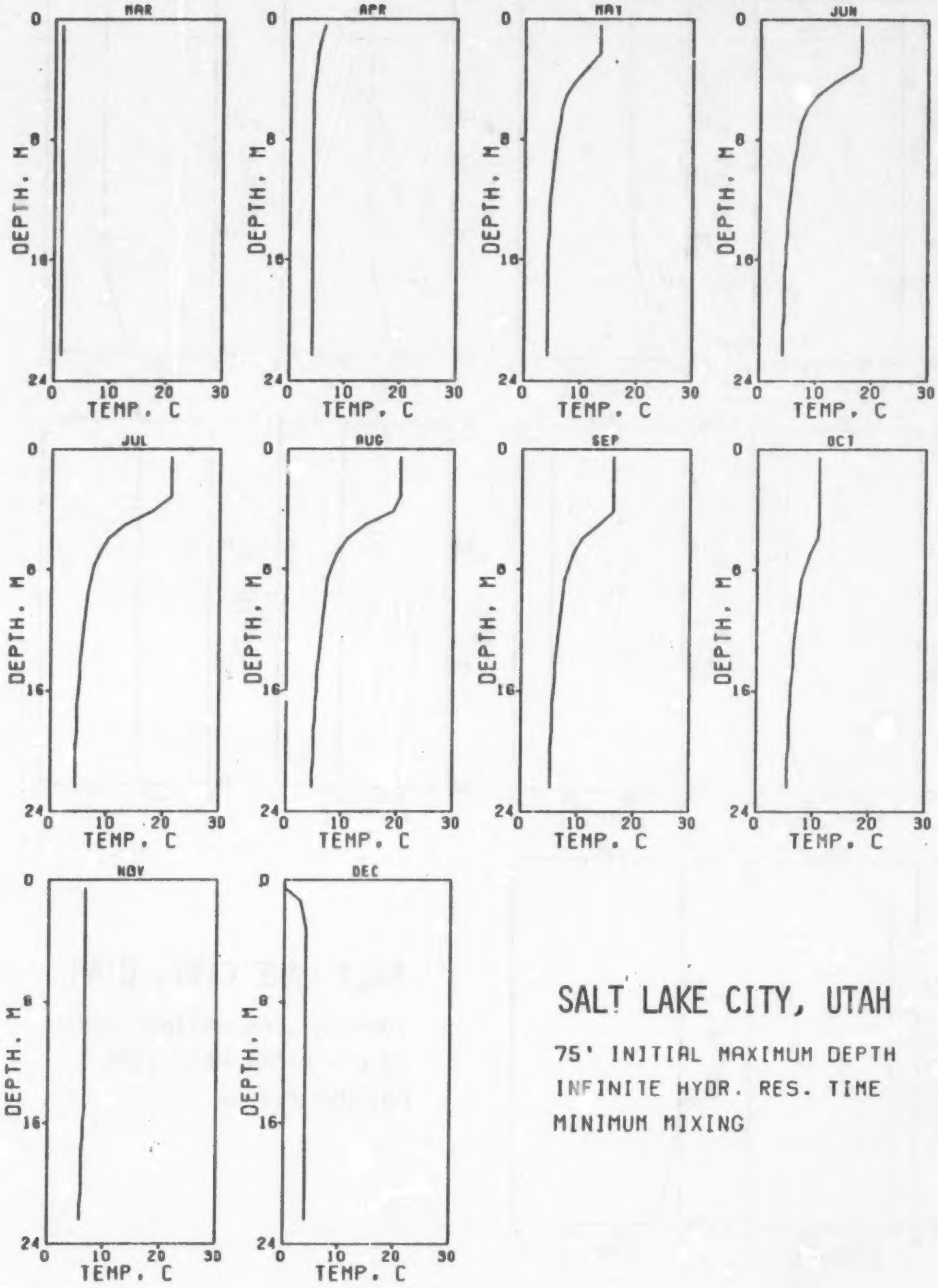






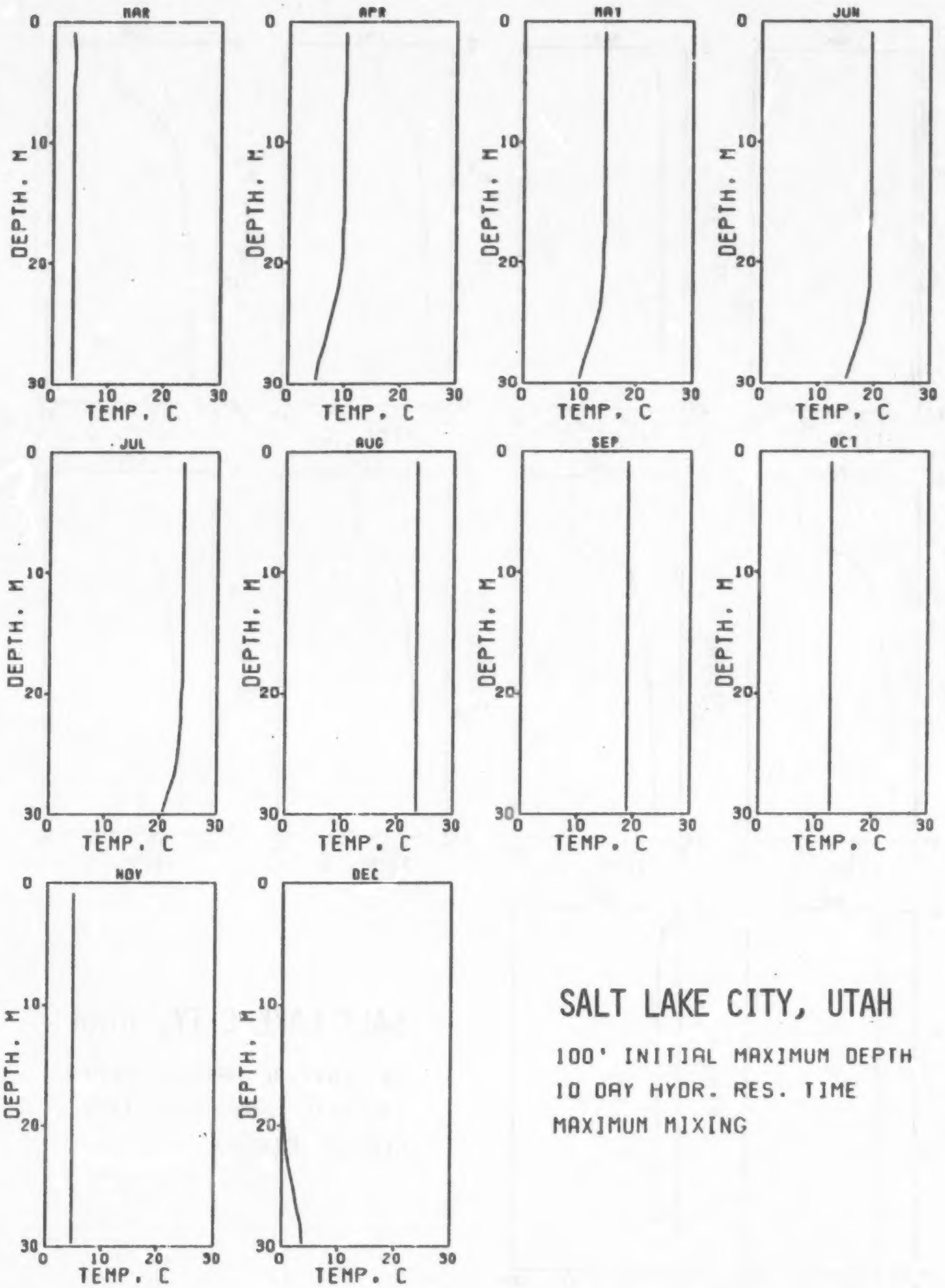
SALT LAKE CITY, UTAH

75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



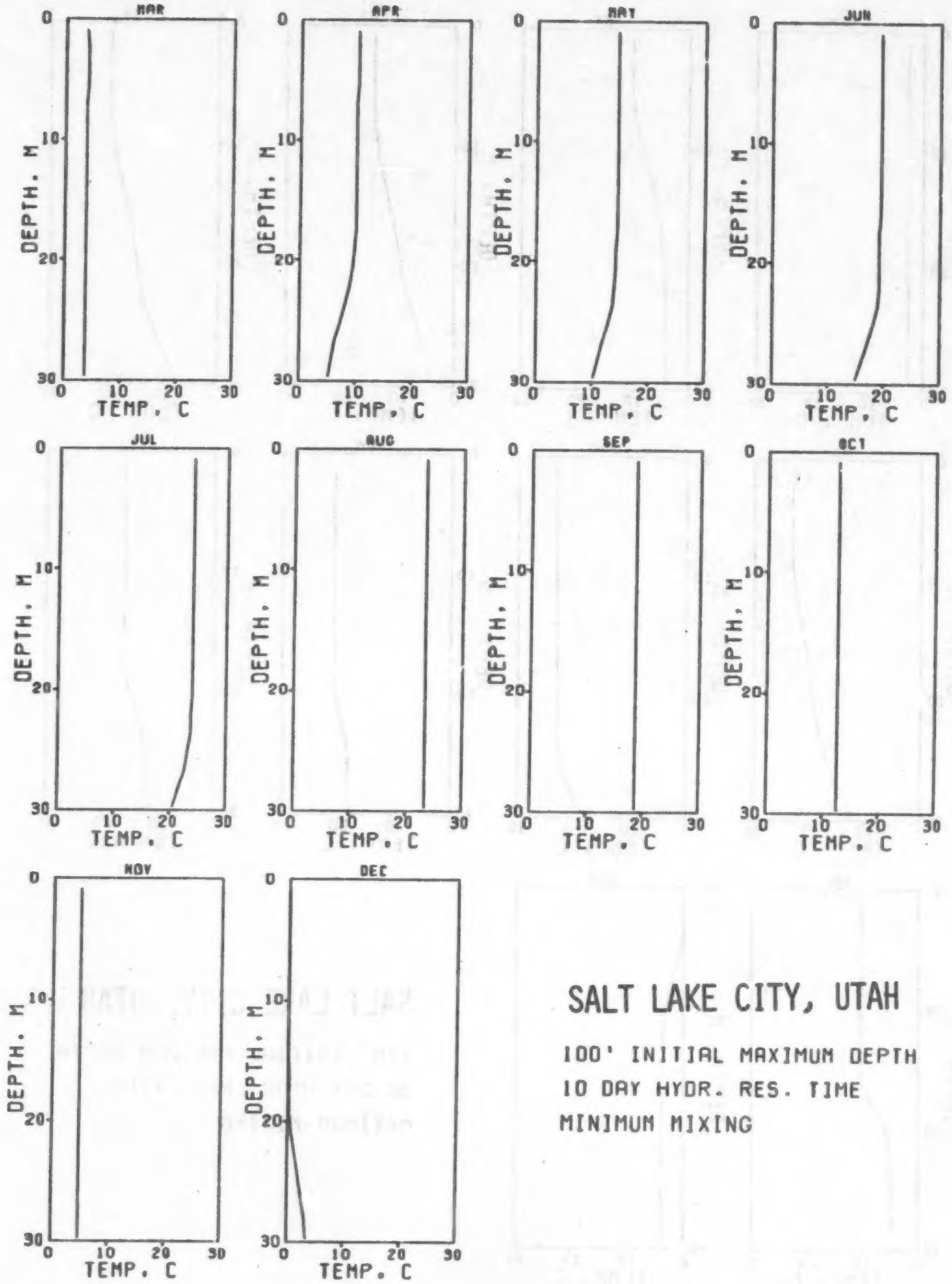
SALT LAKE CITY, UTAH

75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



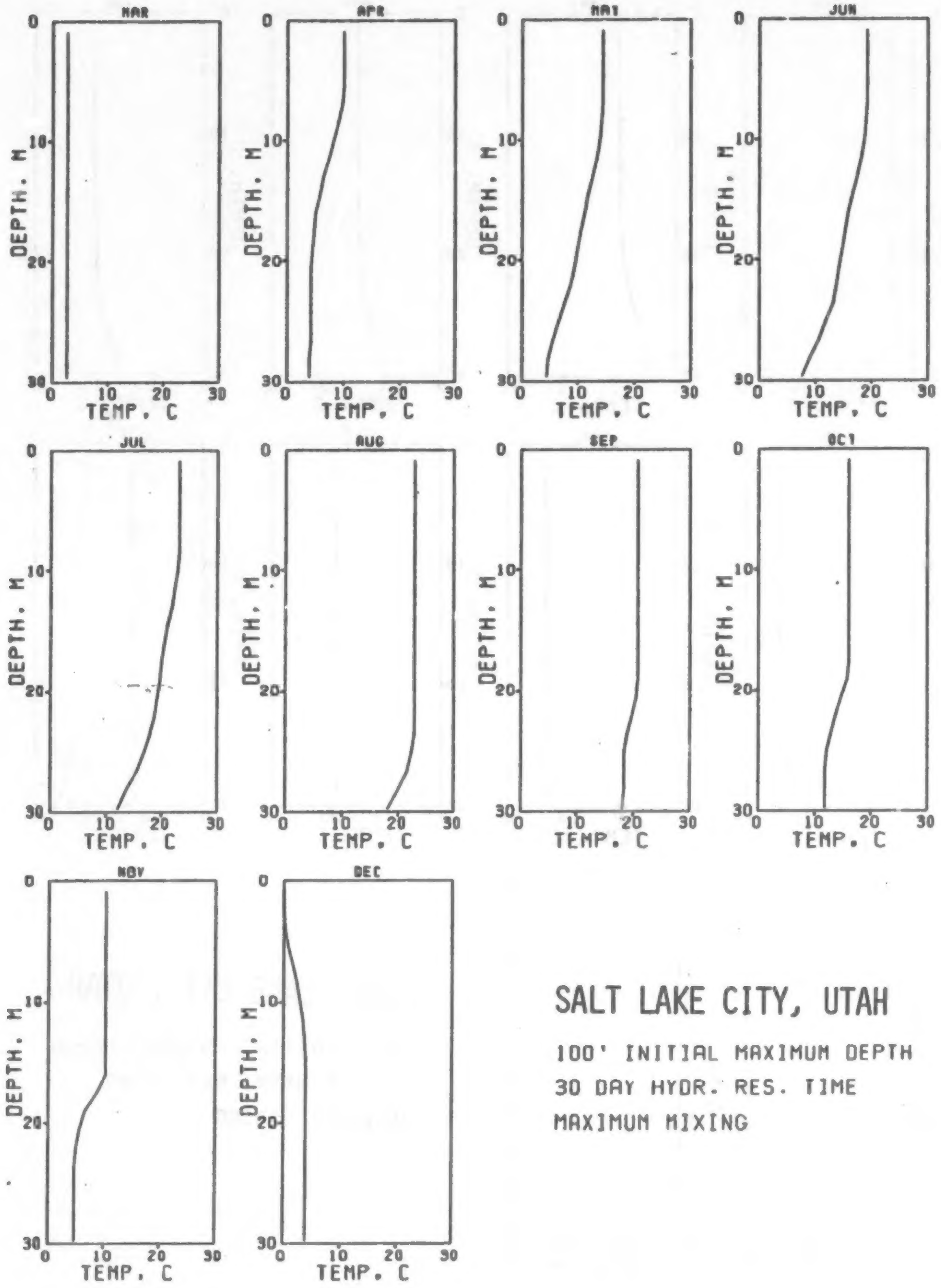
SALT LAKE CITY, UTAH

100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



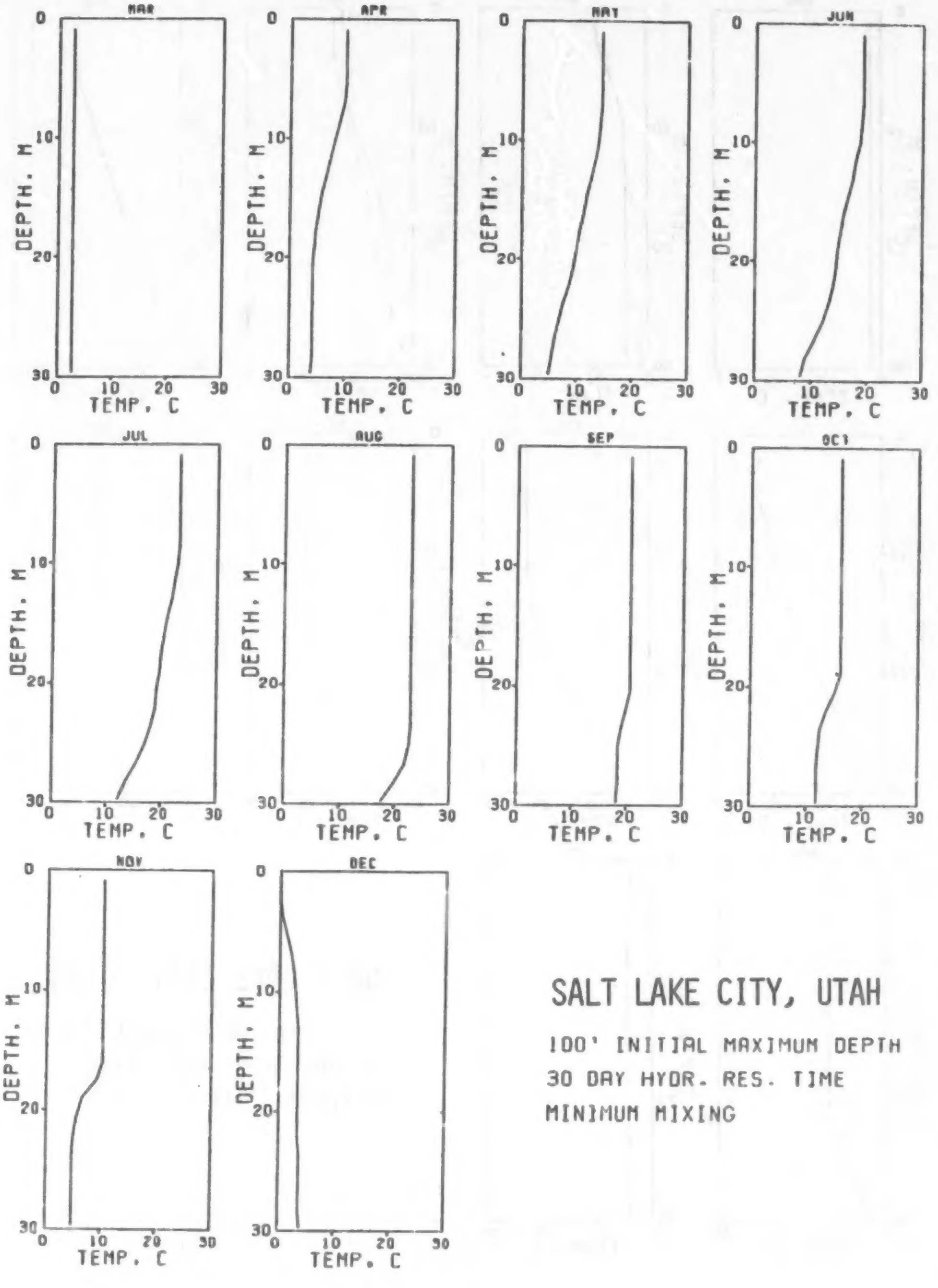
SALT LAKE CITY, UTAH

100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



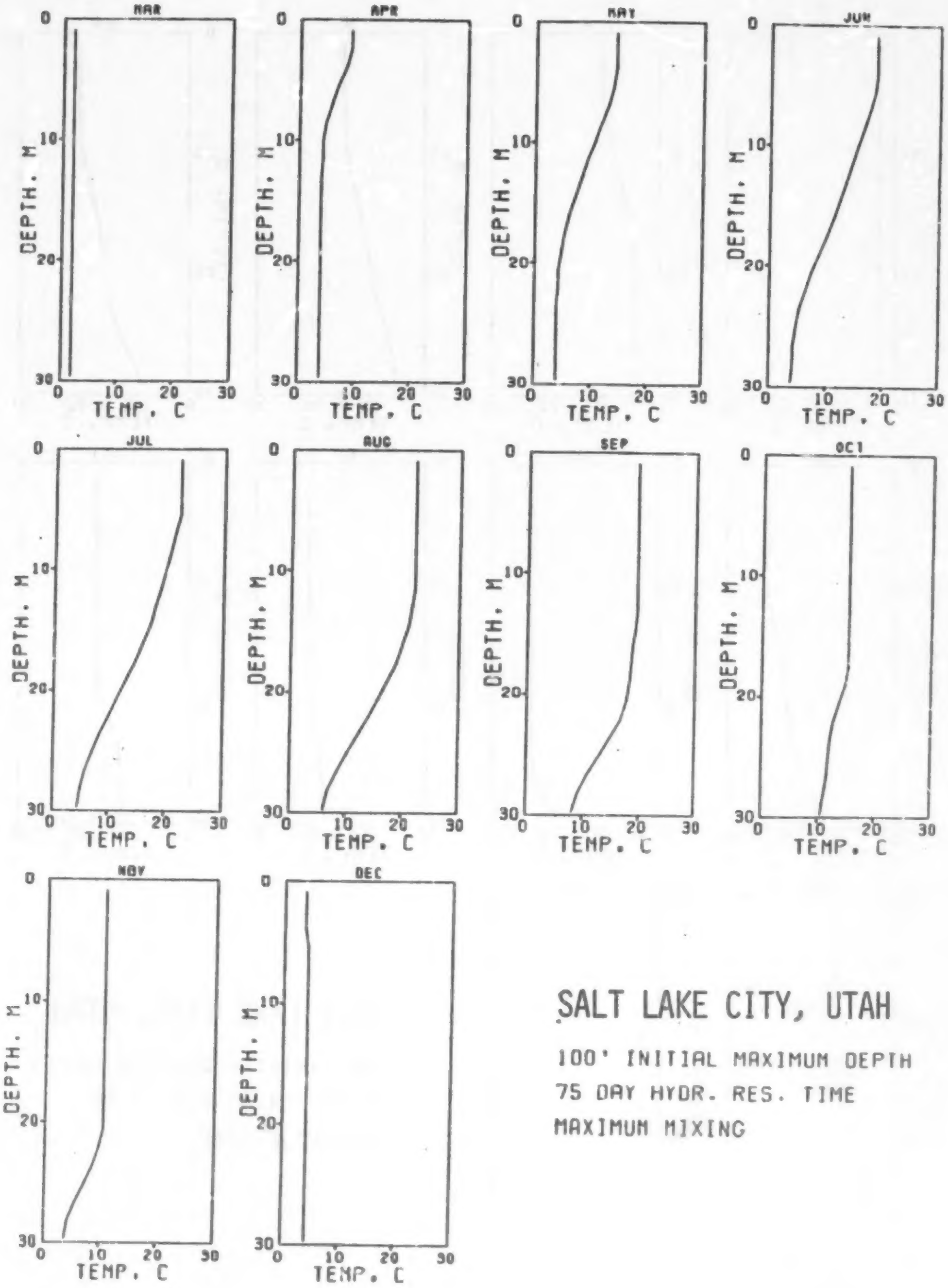
SALT LAKE CITY, UTAH

100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



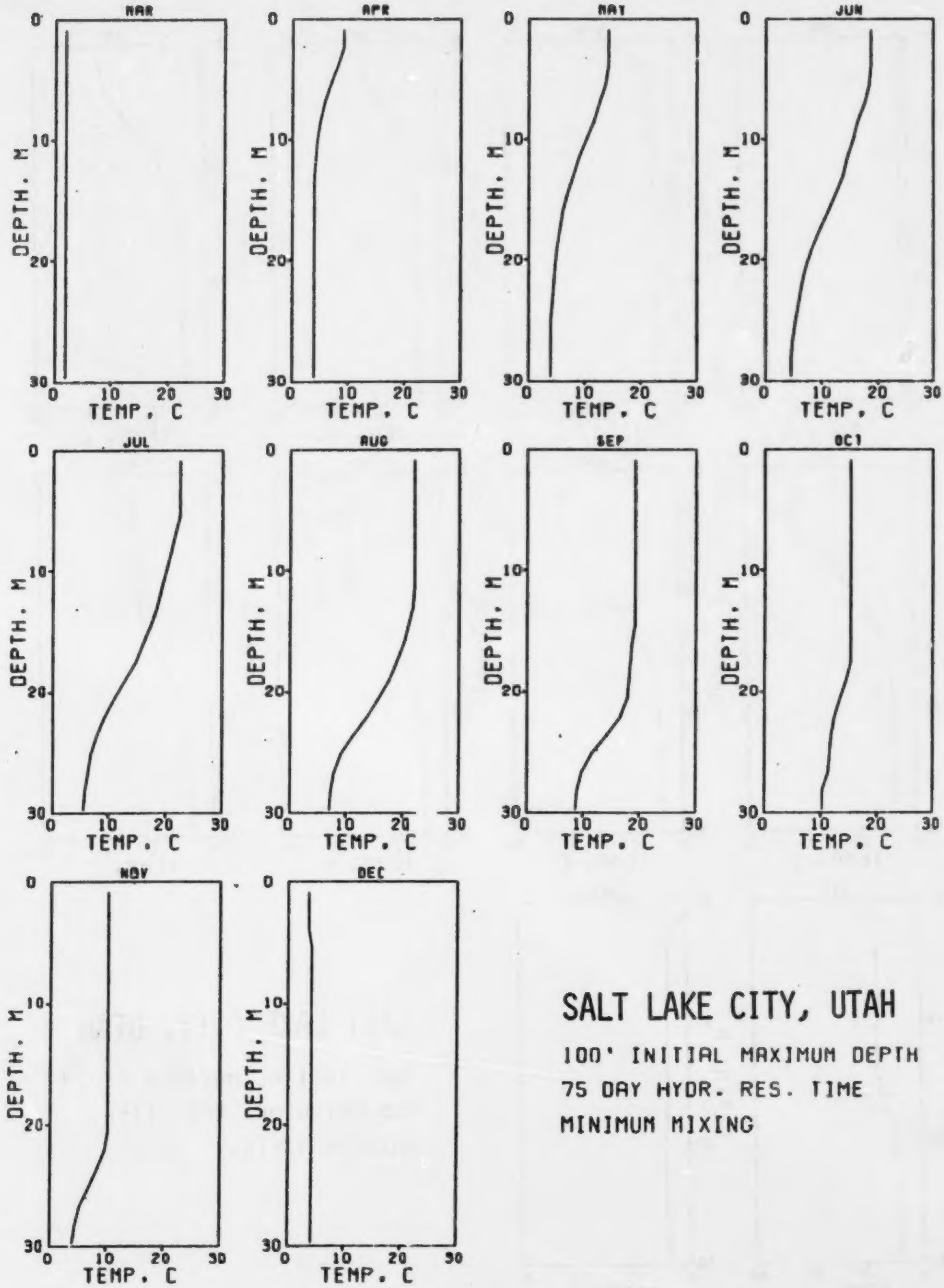
SALT LAKE CITY, UTAH

100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



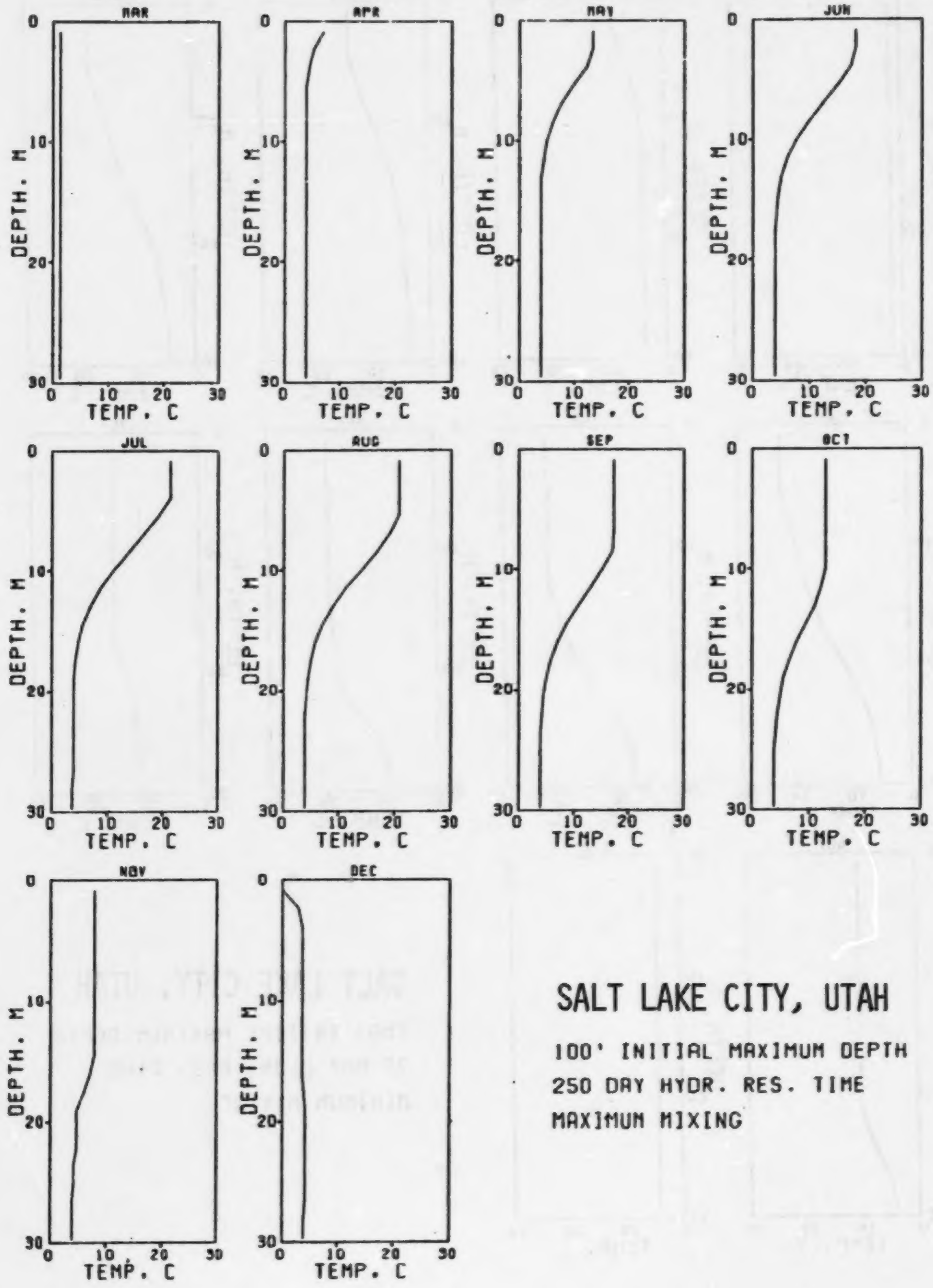
SALT LAKE CITY, UTAH

100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



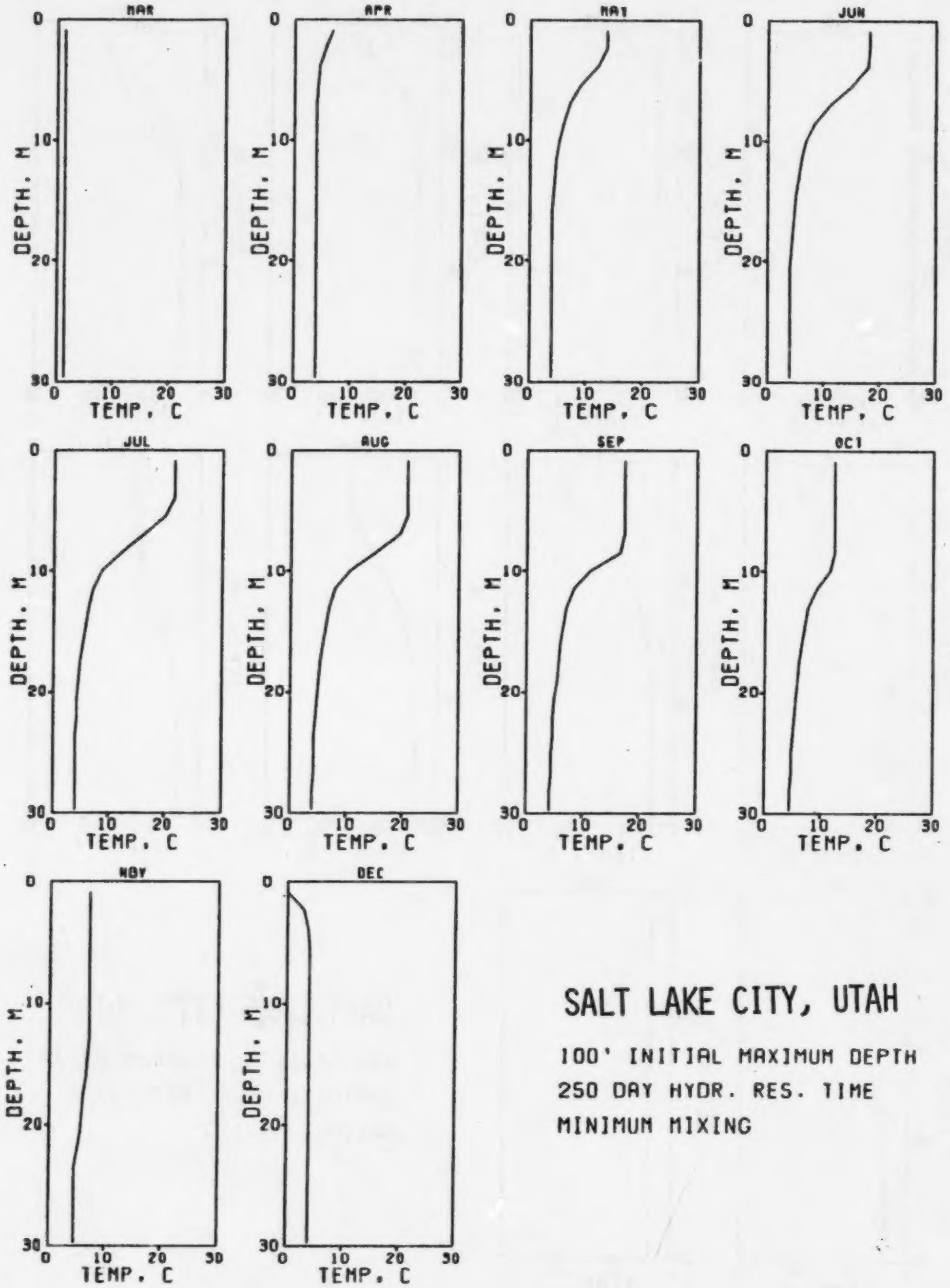
SALT LAKE CITY, UTAH

100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



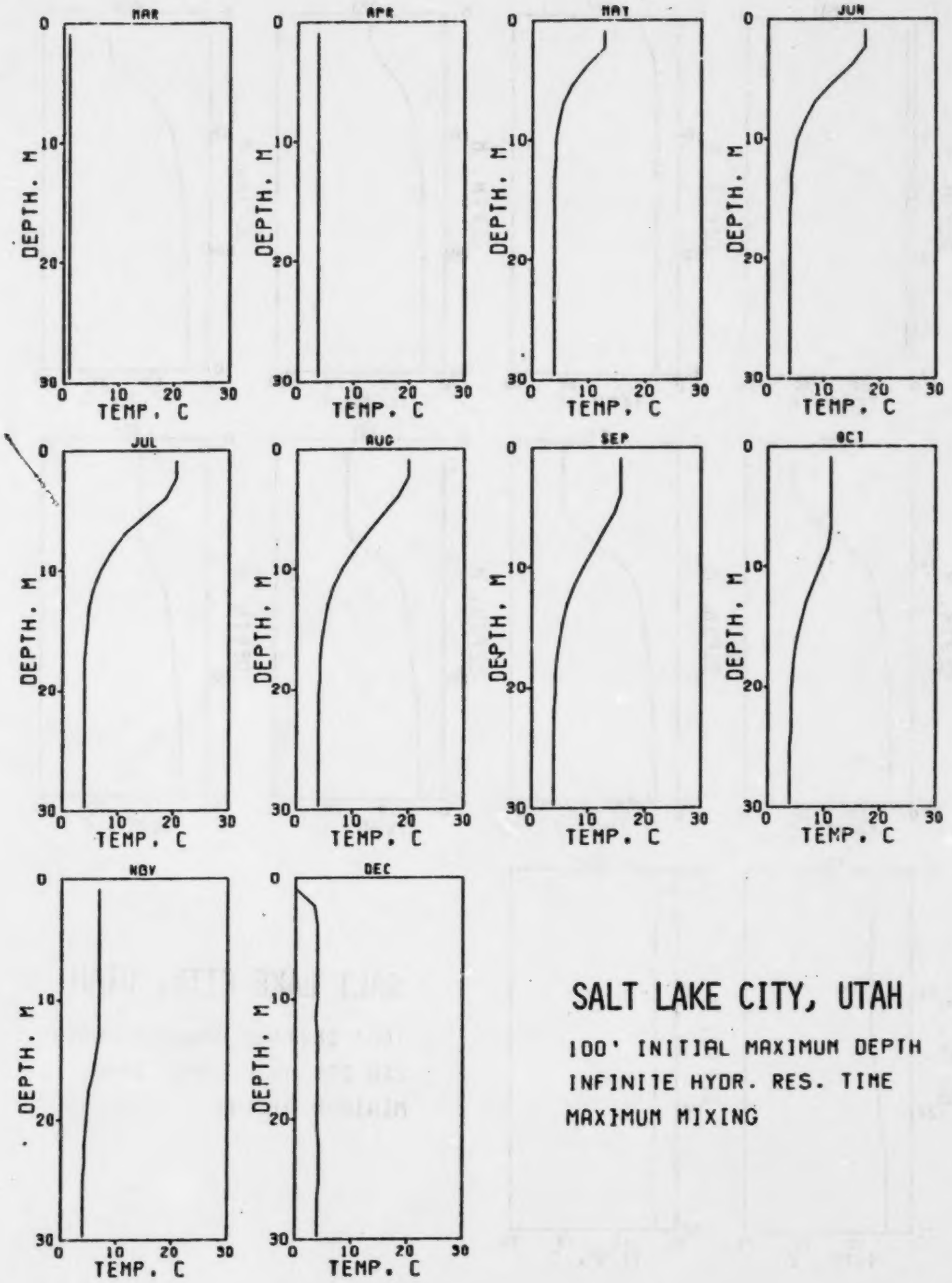
SALT LAKE CITY, UTAH

100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



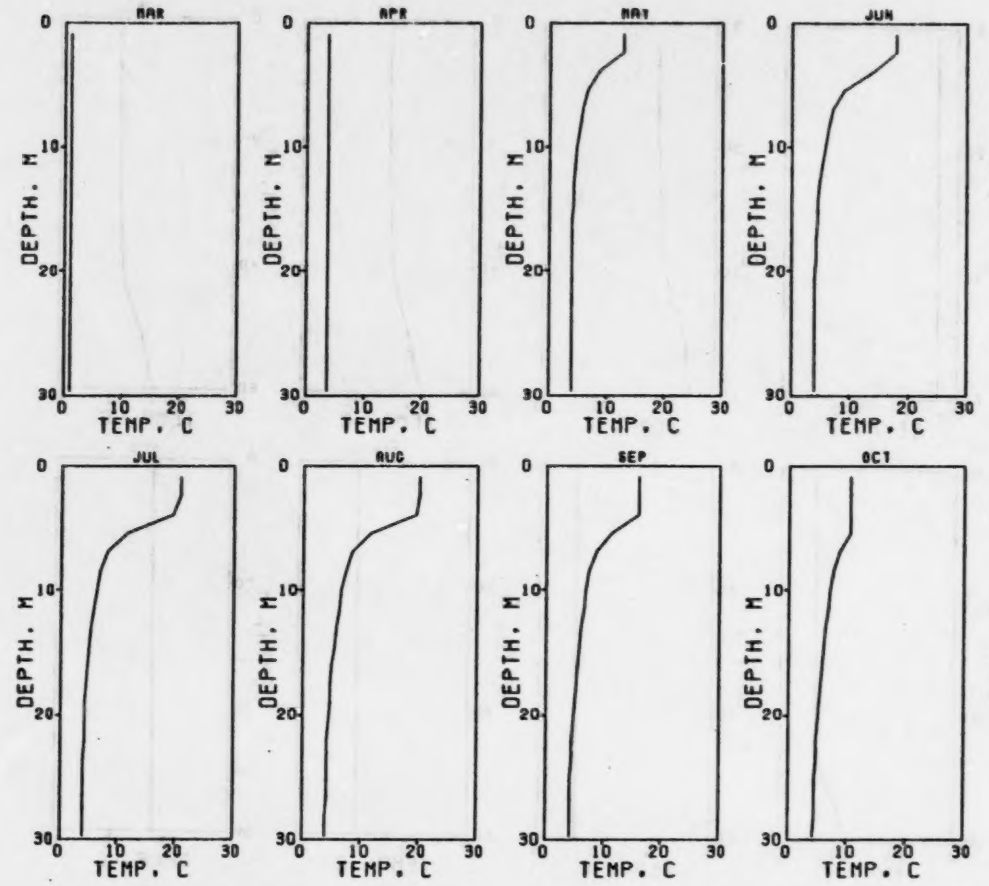
SALT LAKE CITY, UTAH

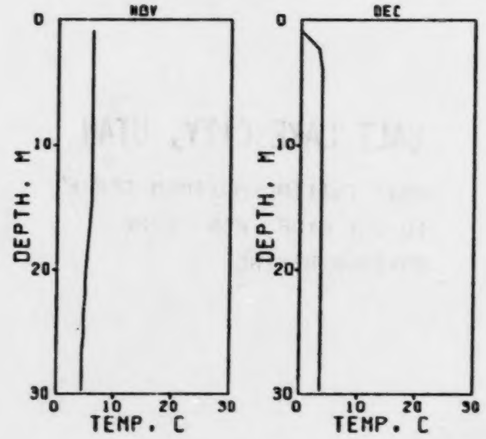
100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



SALT LAKE CITY, UTAH

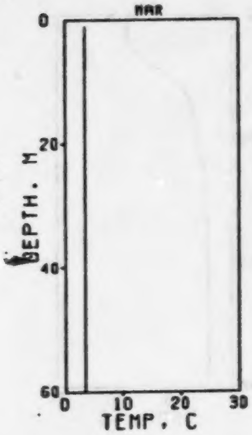
100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING

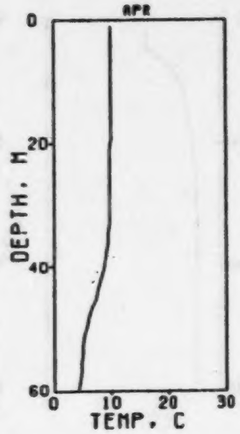


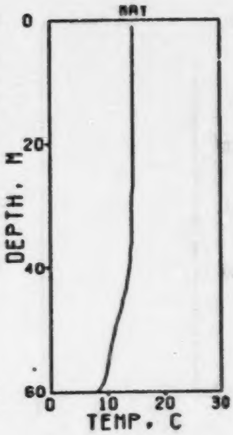


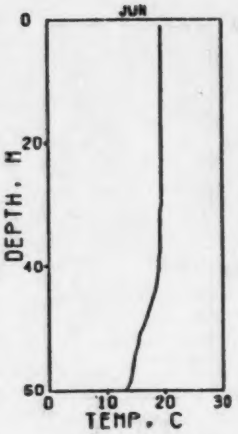
SALT LAKE CITY, UTAH

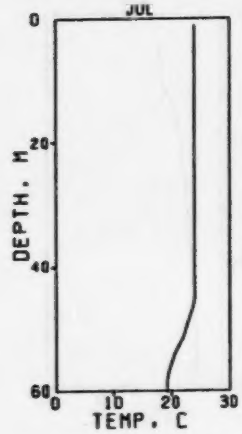
100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING

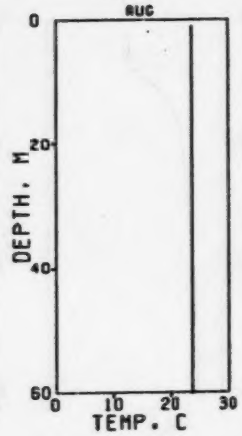


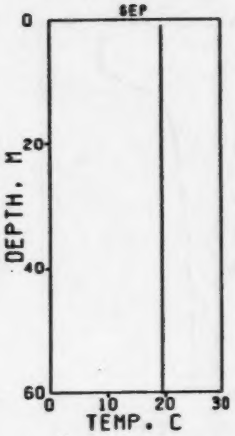


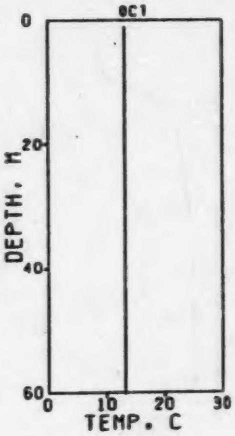


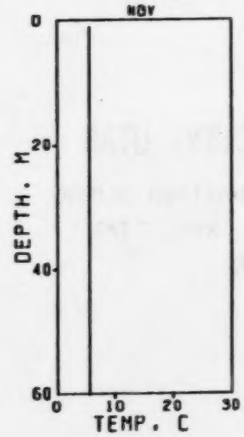


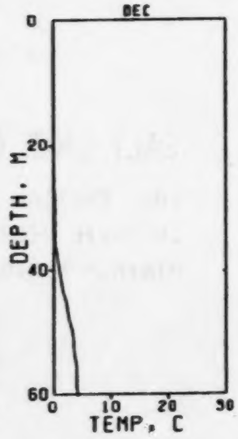






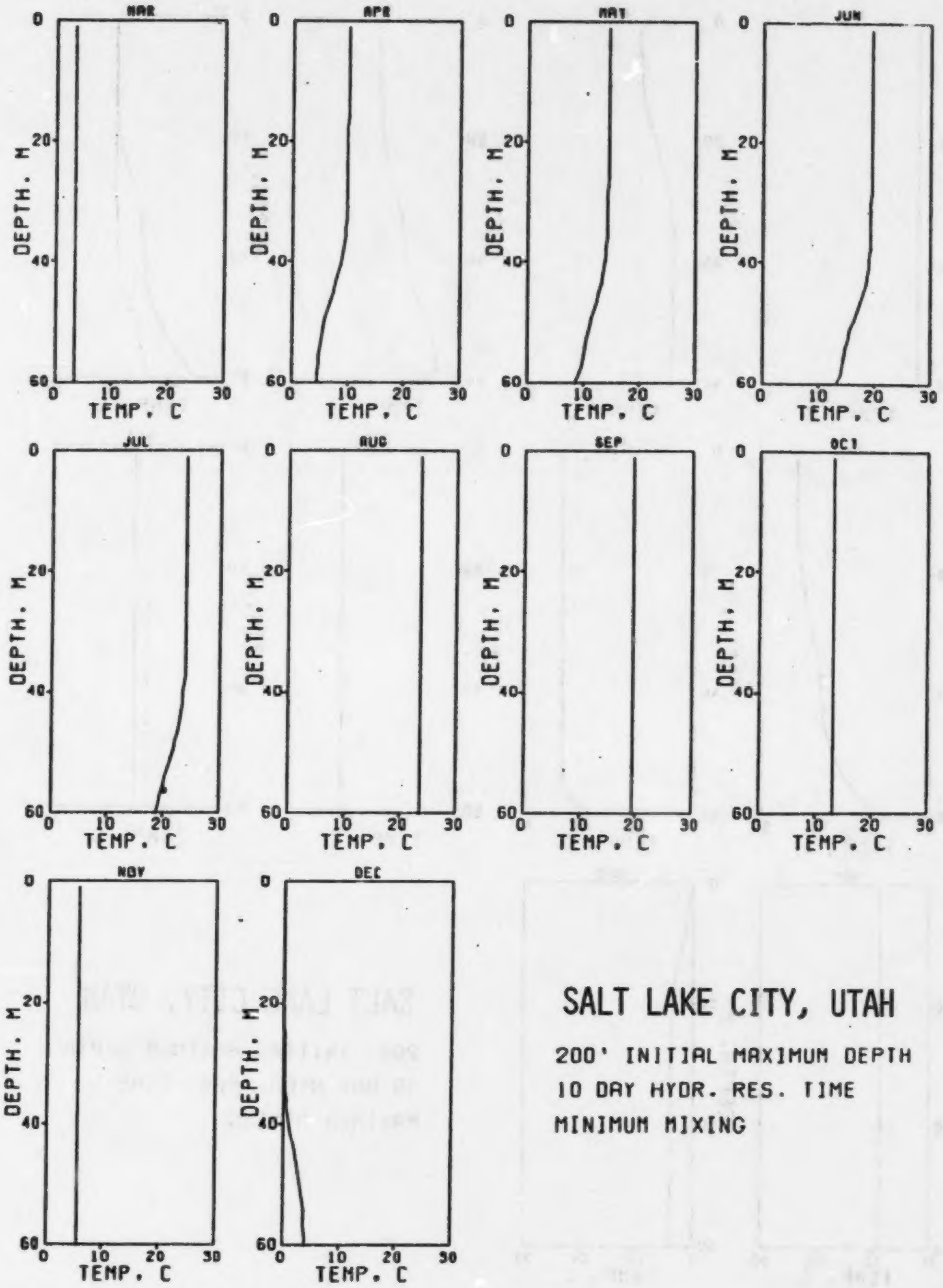






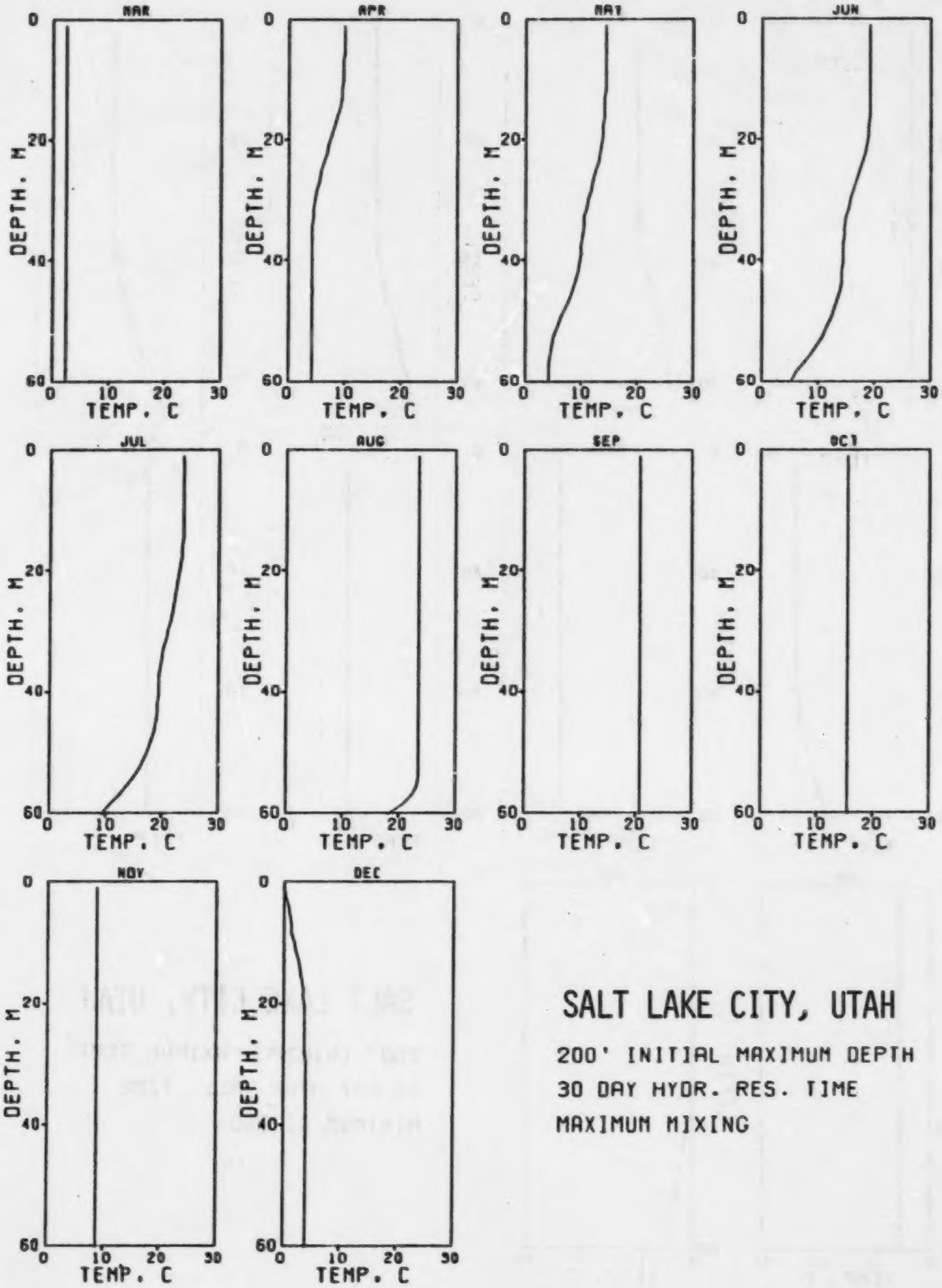
SALT LAKE CITY, UTAH

200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



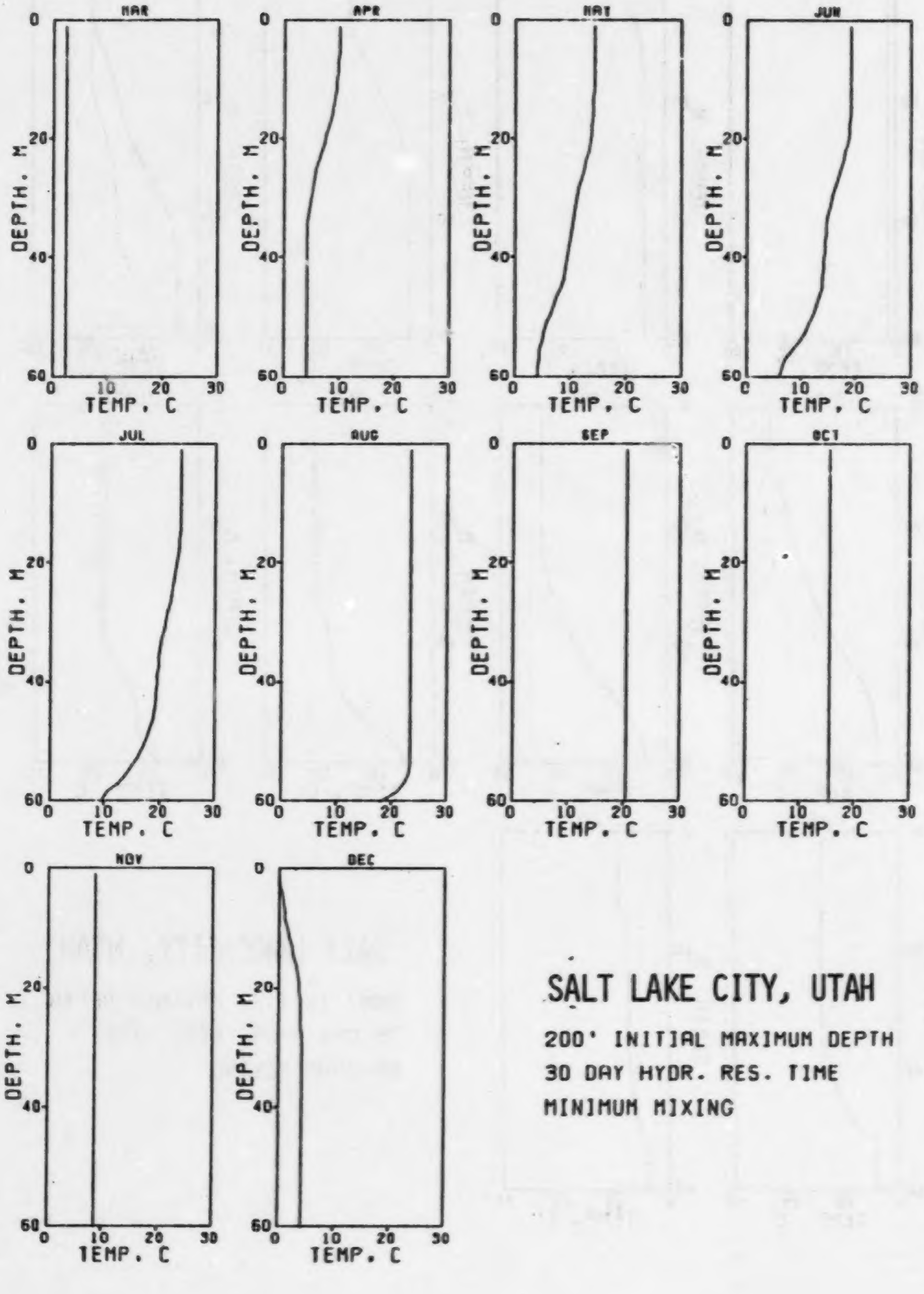
SALT LAKE CITY, UTAH

200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



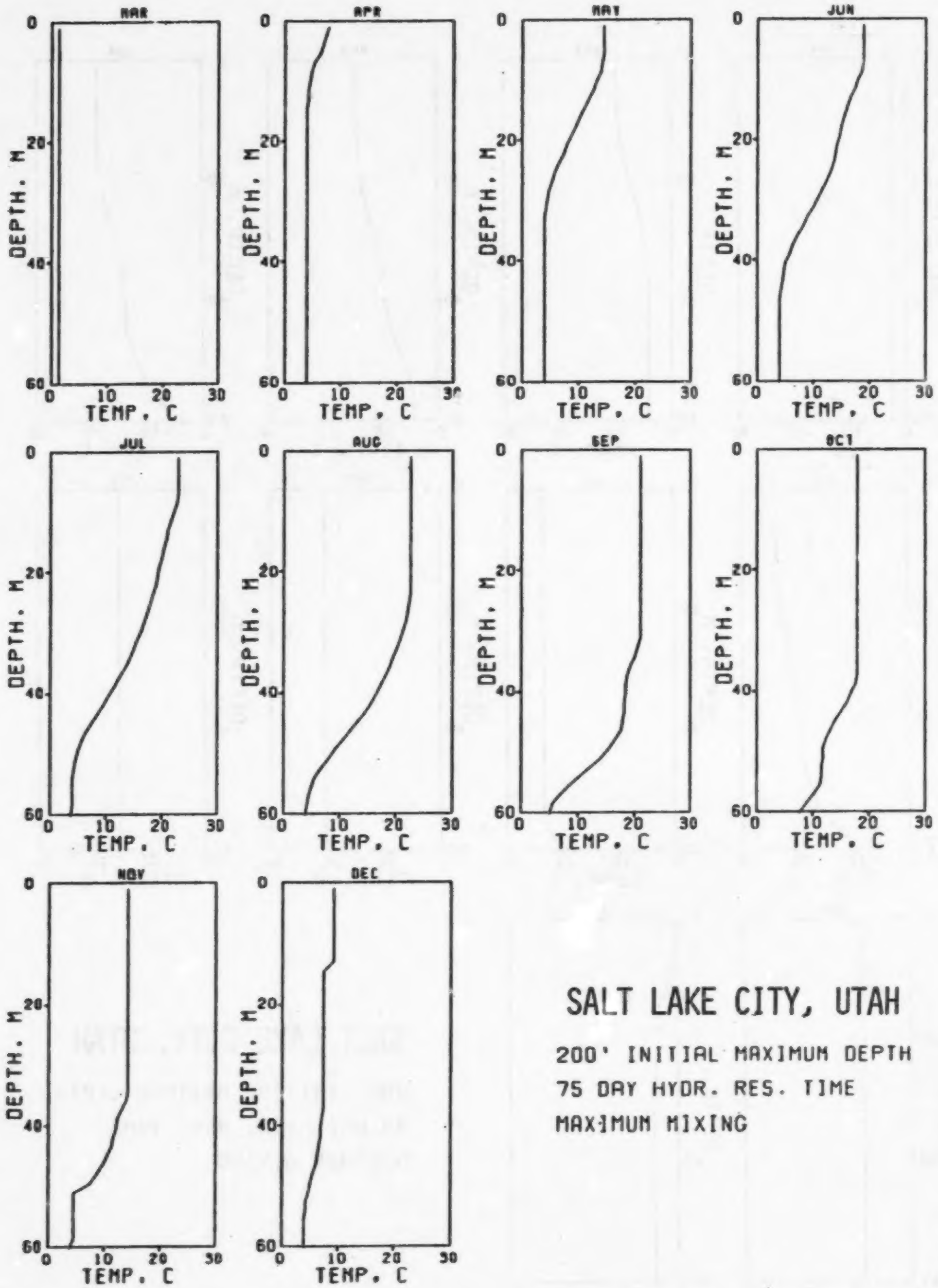
SALT LAKE CITY, UTAH

200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



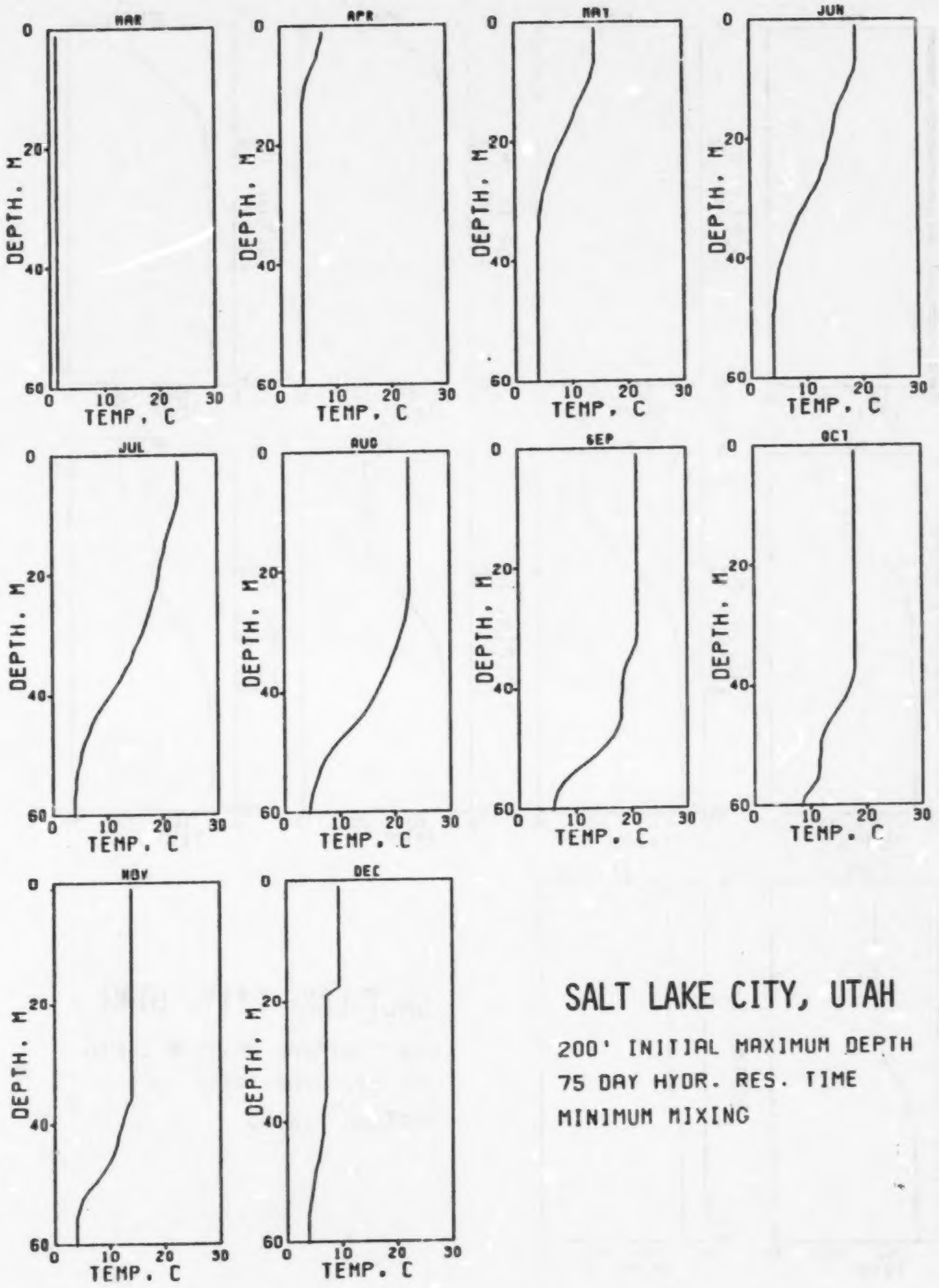
SALT LAKE CITY, UTAH

200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



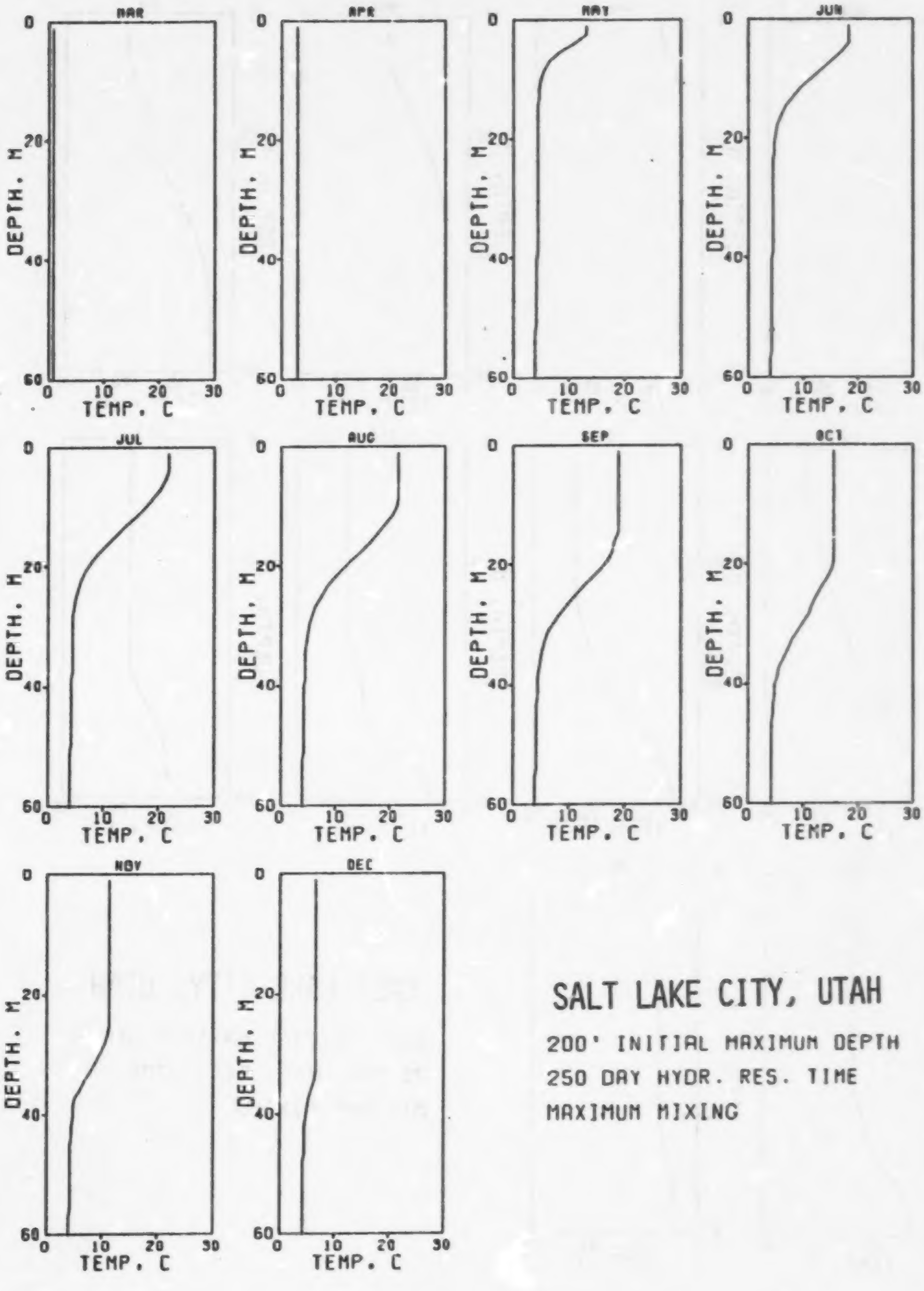
SALT LAKE CITY, UTAH

200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



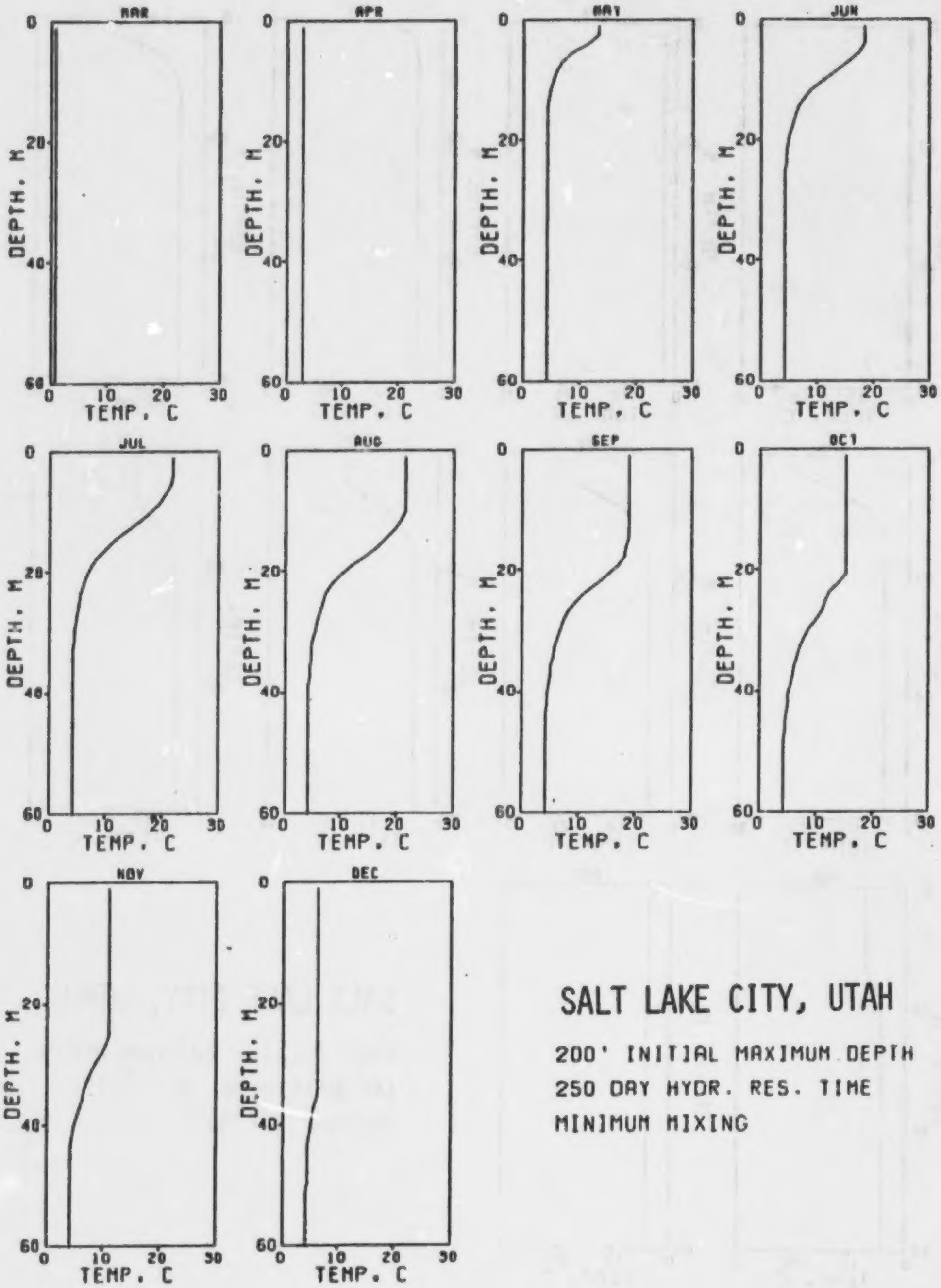
SALT LAKE CITY, UTAH

200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



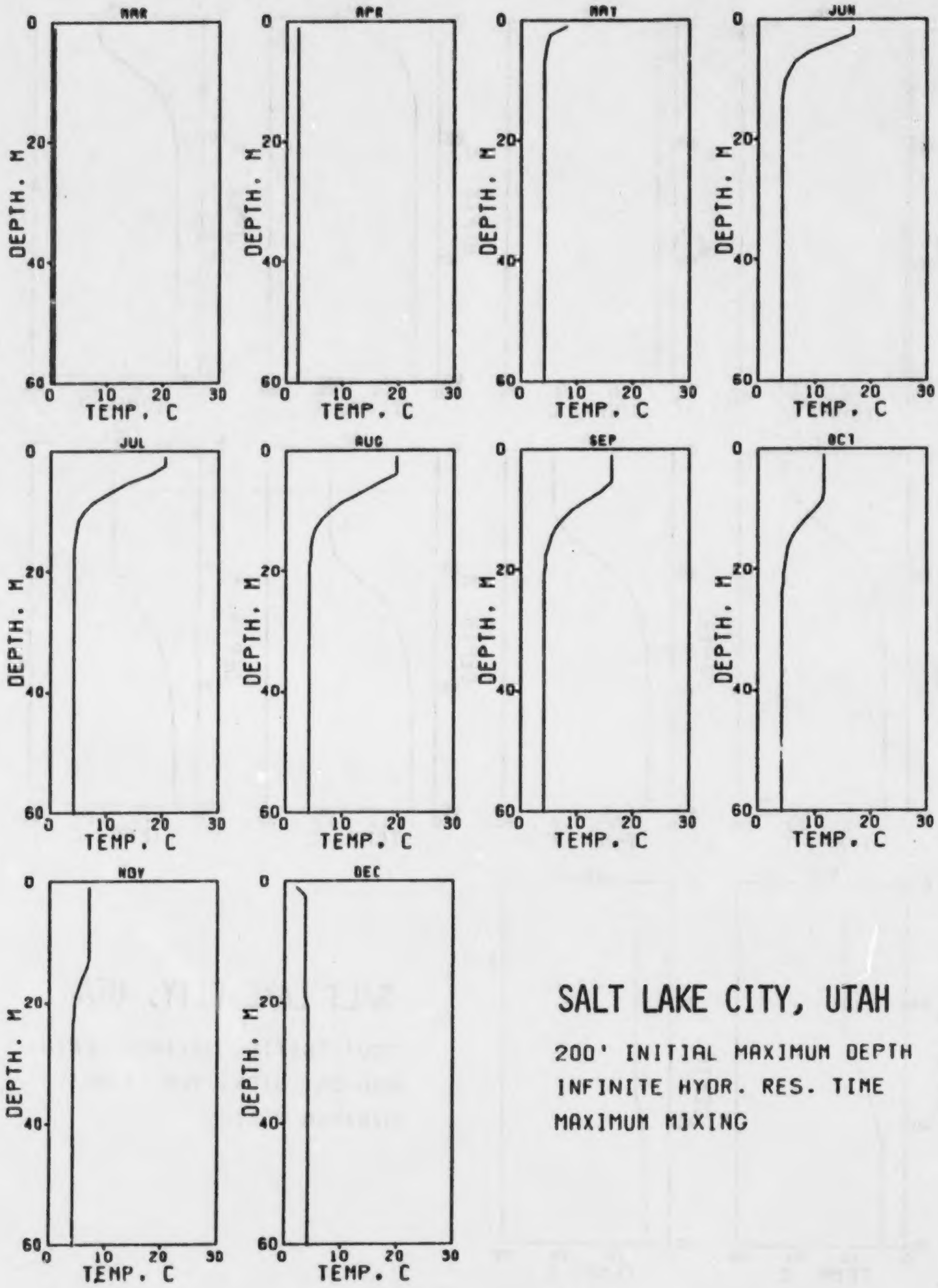
SALT LAKE CITY, UTAH

200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



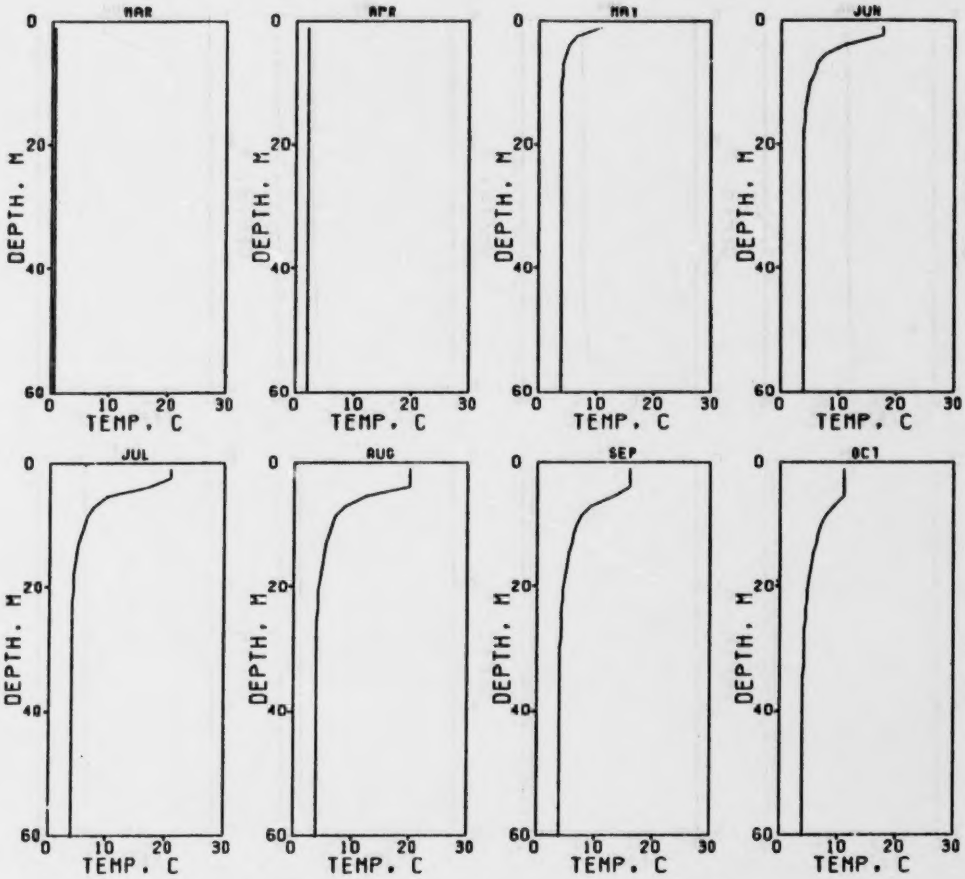
SALT LAKE CITY, UTAH

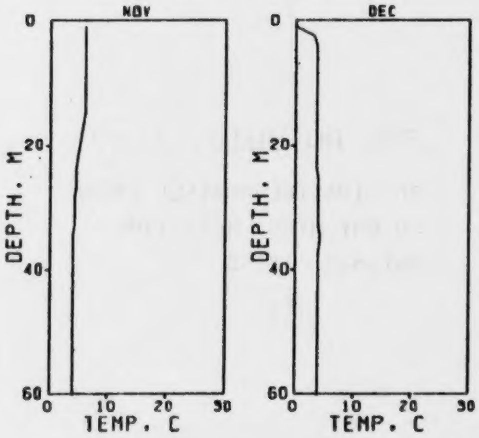
200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



SALT LAKE CITY, UTAH

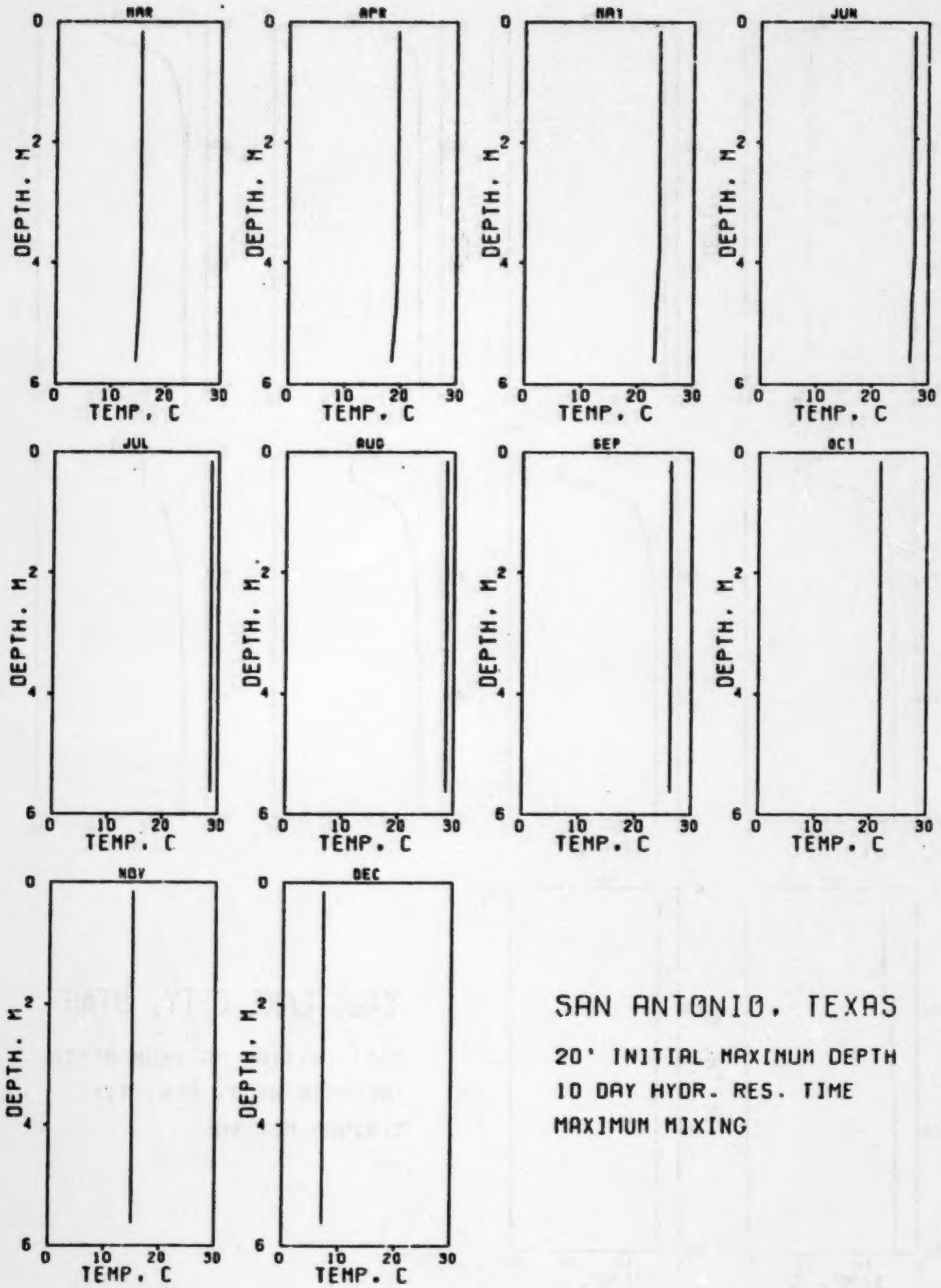
200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING





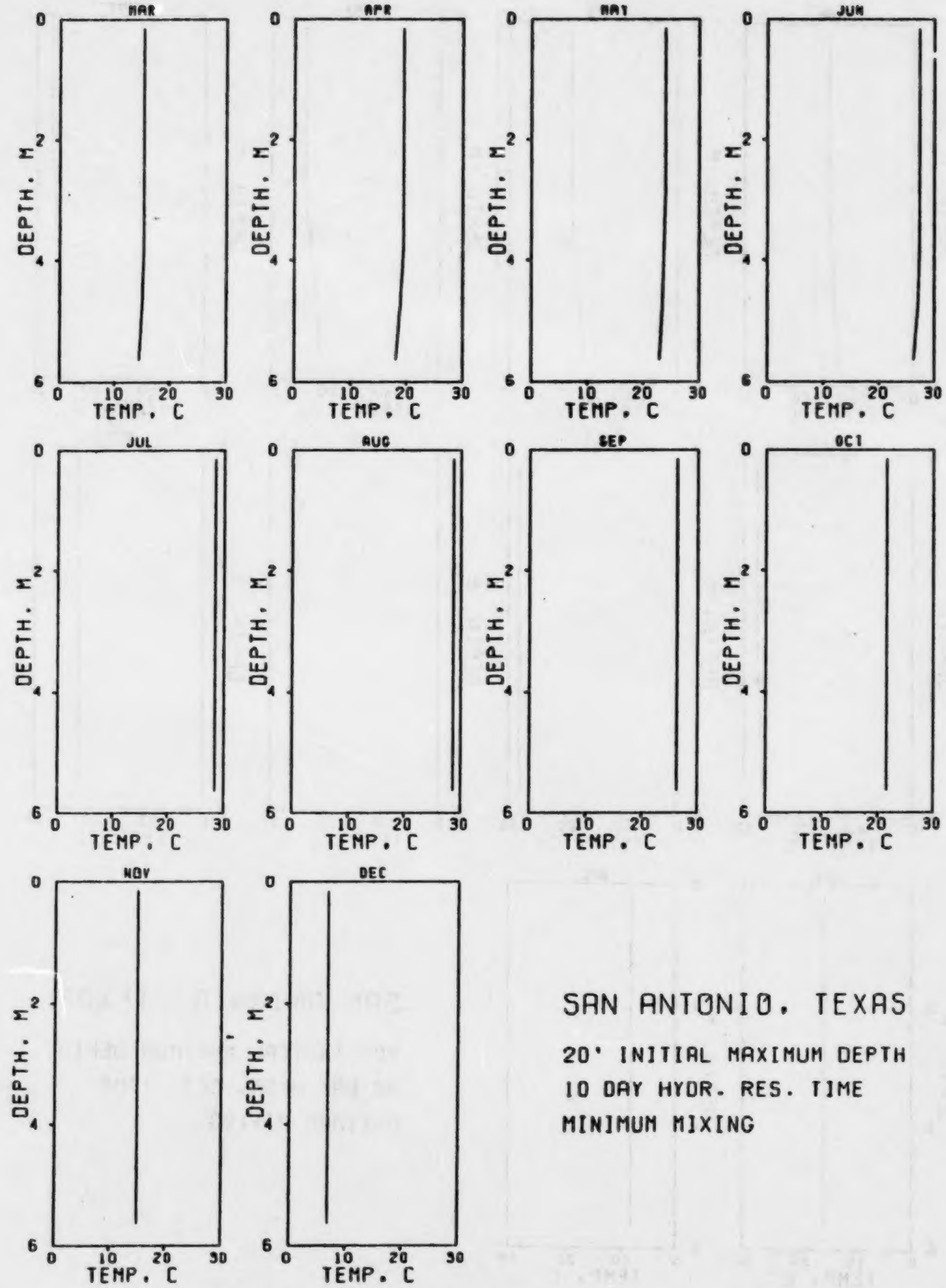
SALT LAKE CITY, UTAH

200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



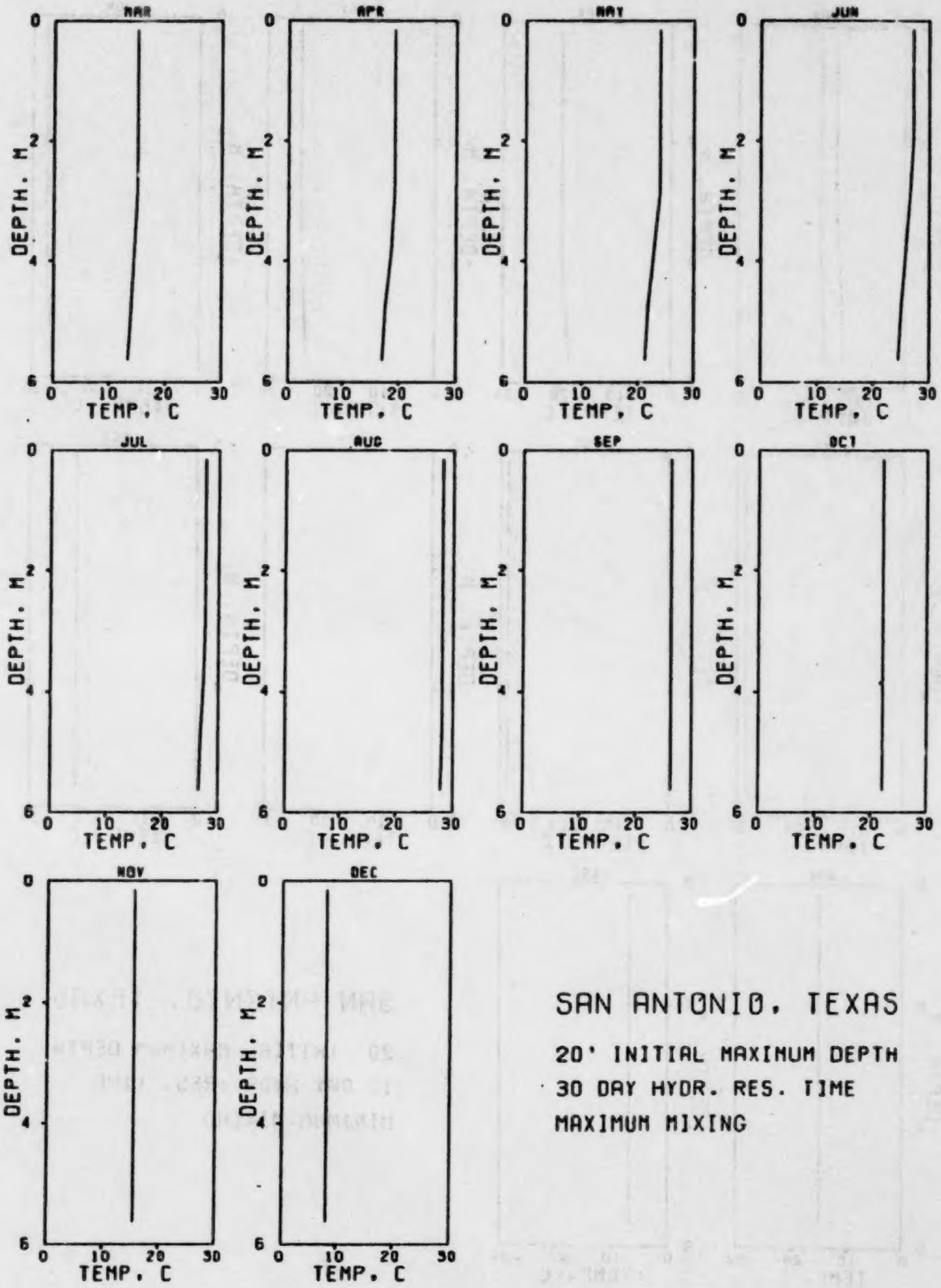
SAN ANTONIO, TEXAS

20' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



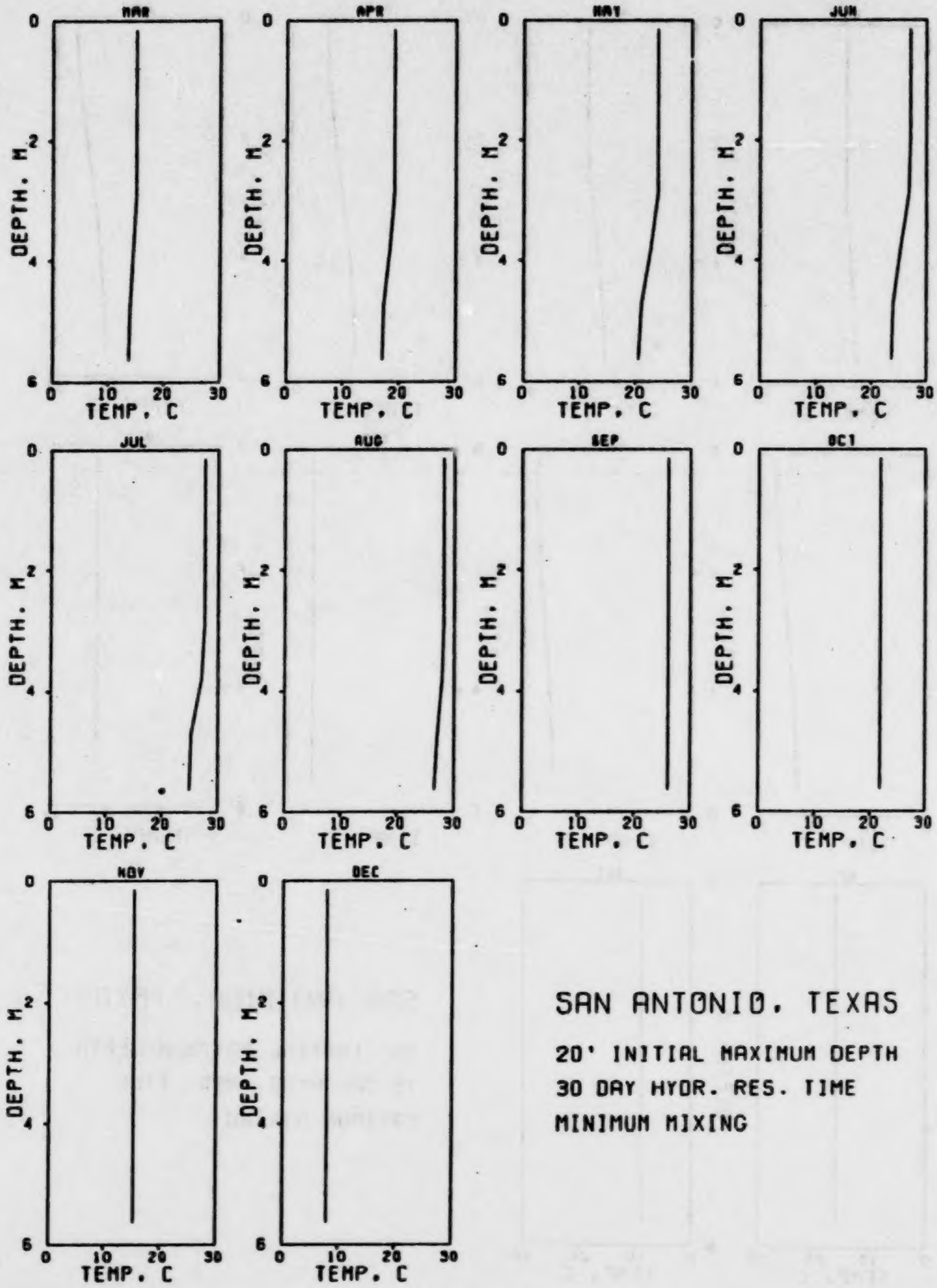
SAN ANTONIO, TEXAS

20' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



SAN ANTONIO, TEXAS

20' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING

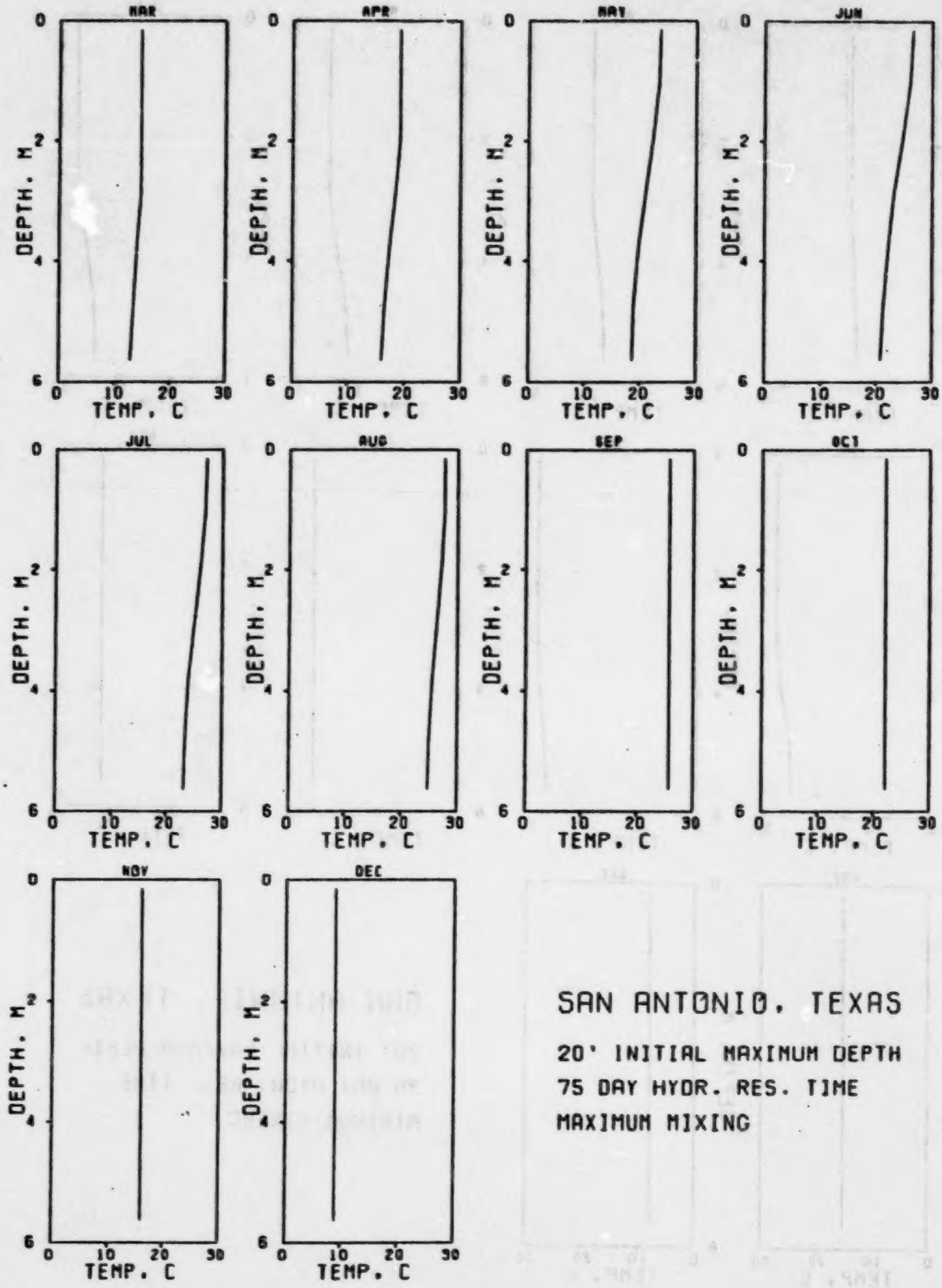


SAN ANTONIO, TEXAS

20' INITIAL MAXIMUM DEPTH

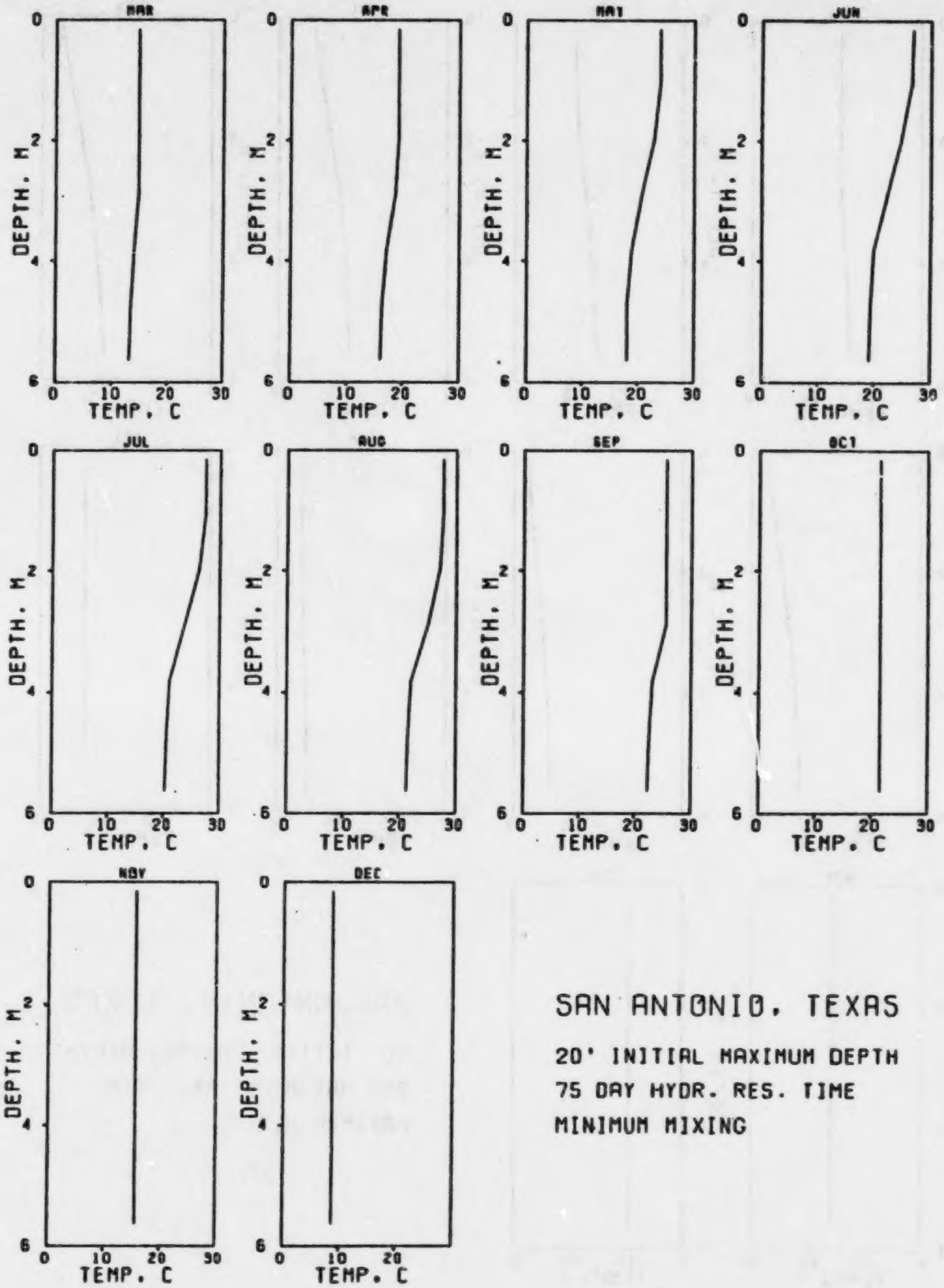
30 DAY HYDR. RES. TIME

MINIMUM MIXING



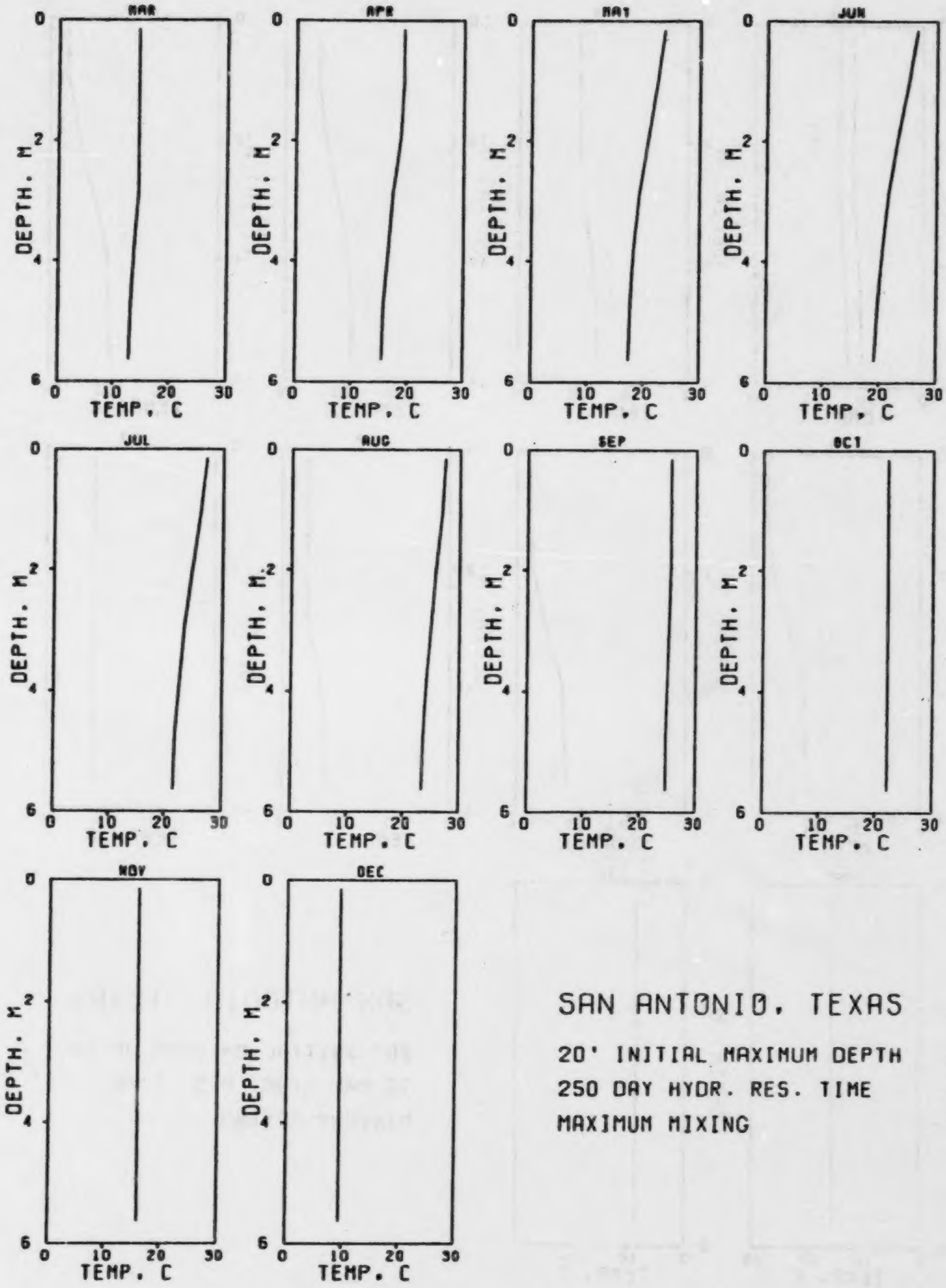
SAN ANTONIO, TEXAS

20' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



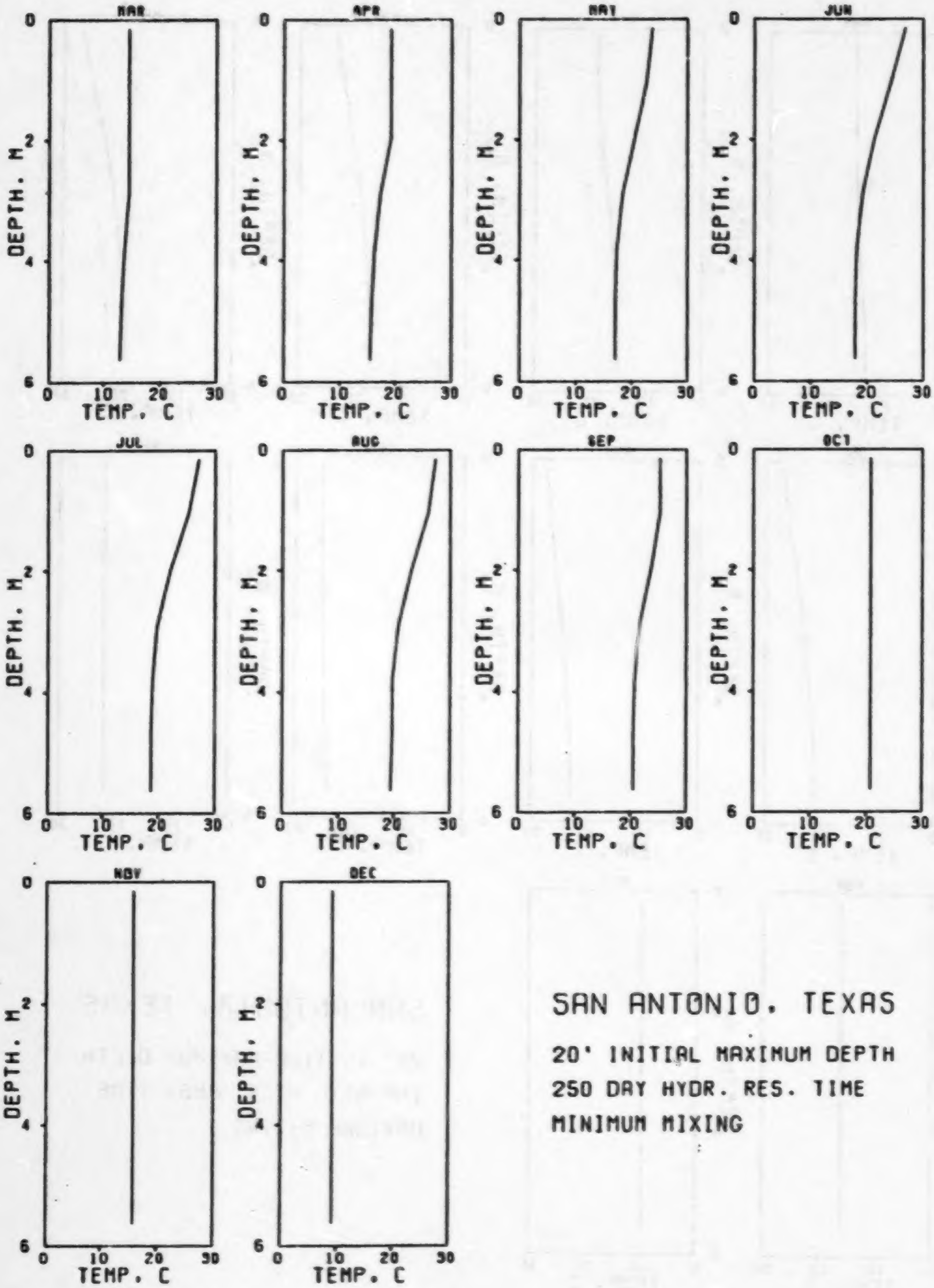
SAN ANTONIO, TEXAS

20' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



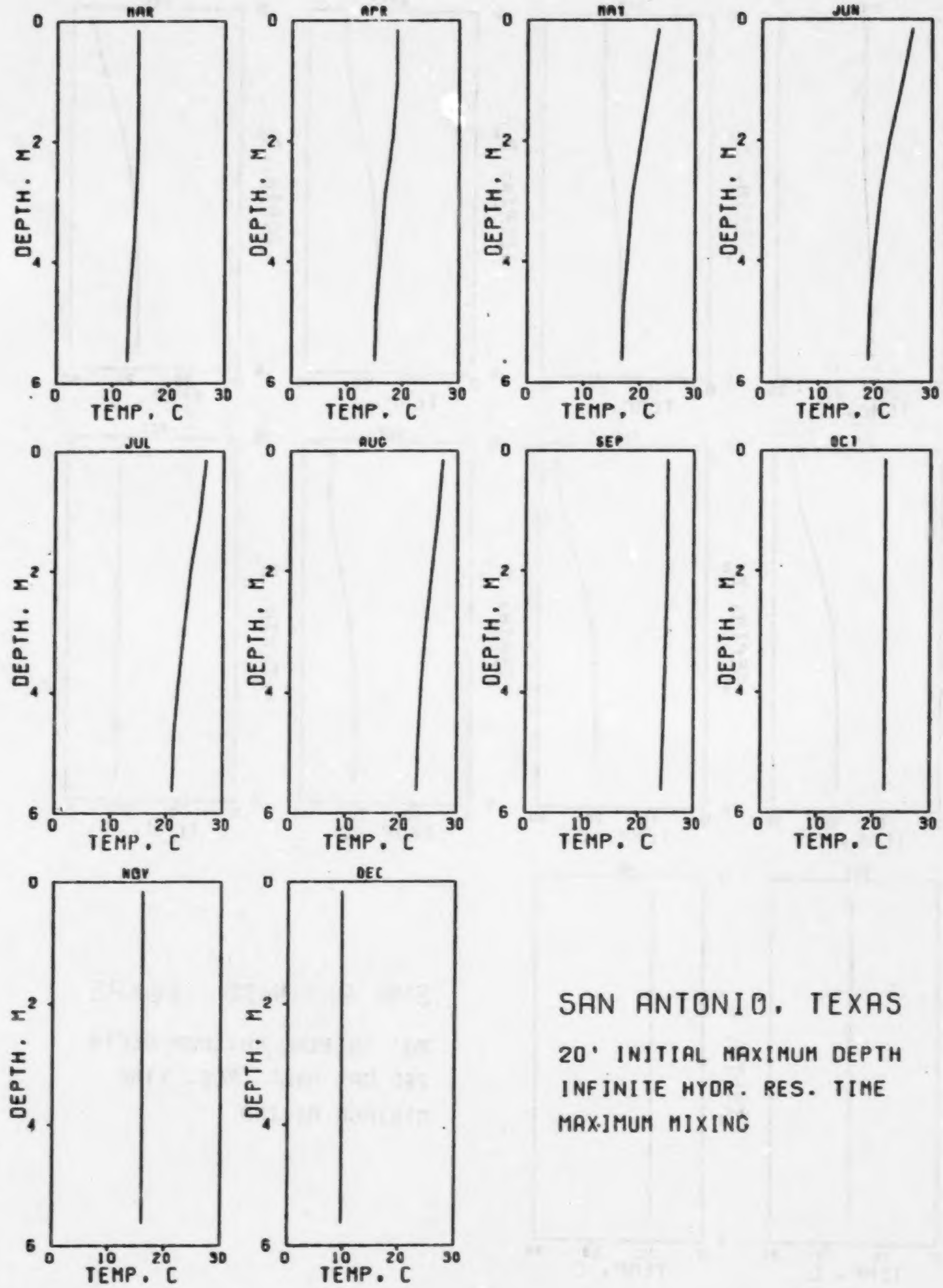
SAN ANTONIO, TEXAS

20' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



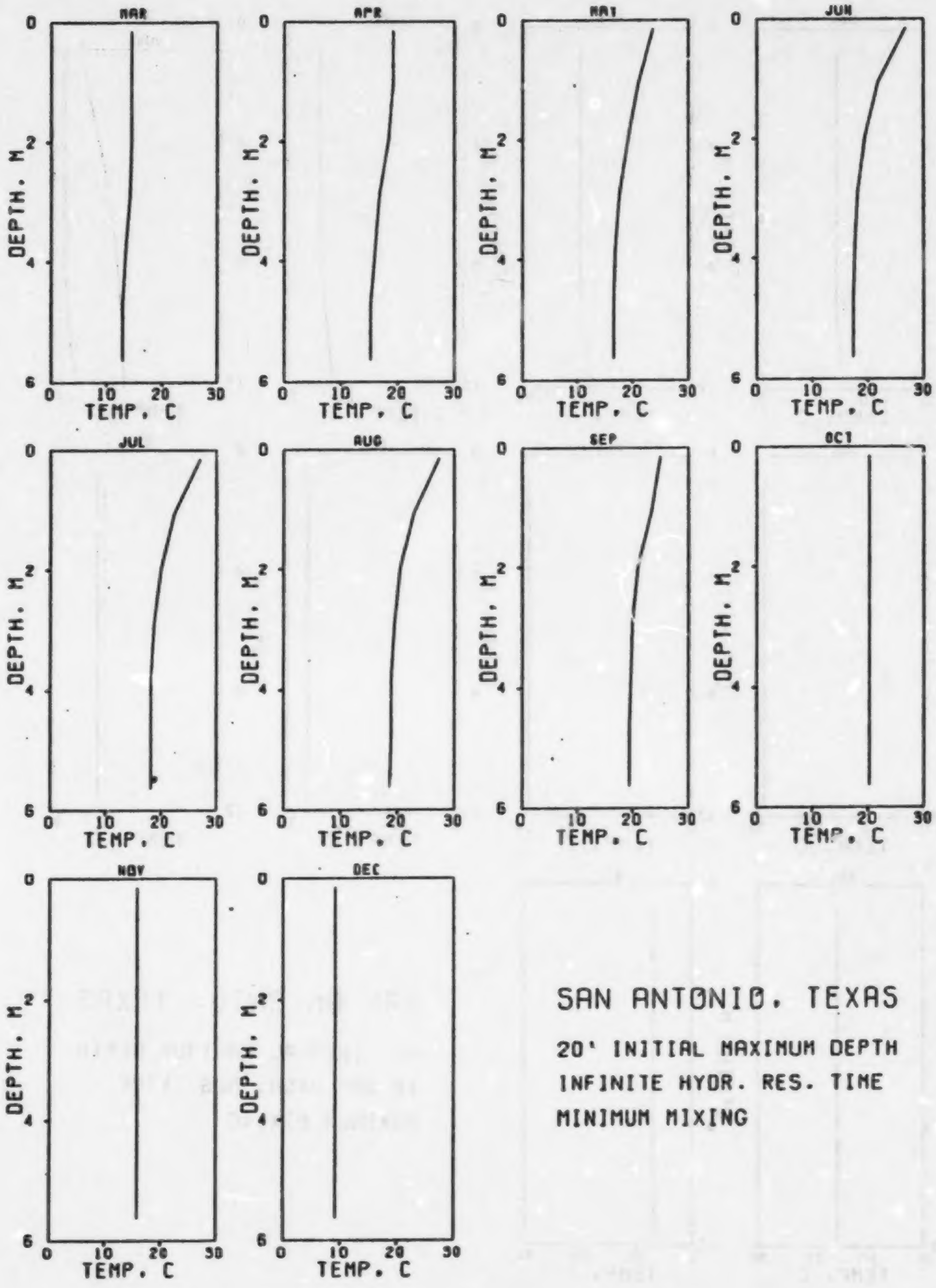
SAN ANTONIO, TEXAS

20' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING

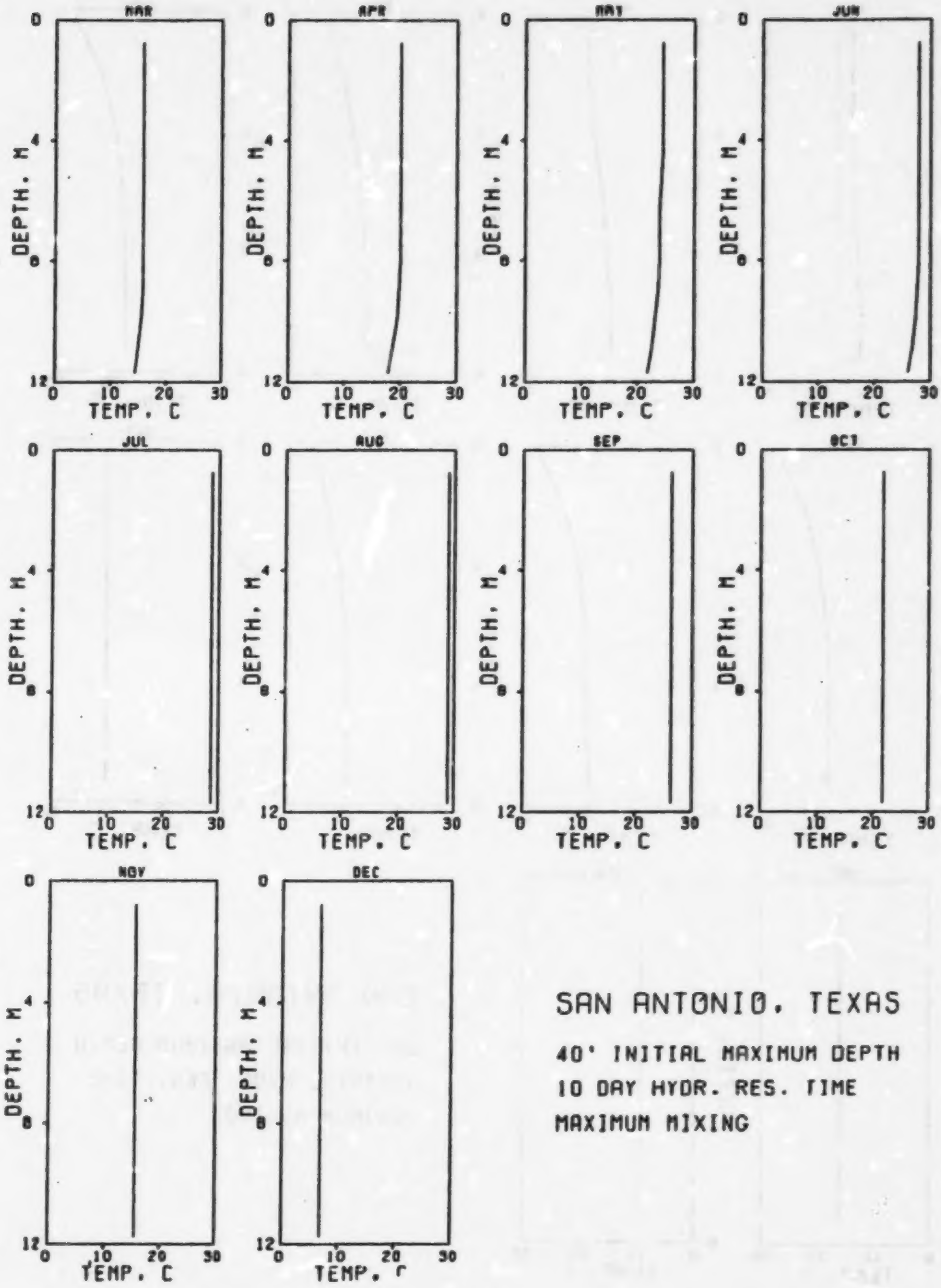


SAN ANTONIO, TEXAS

20' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING

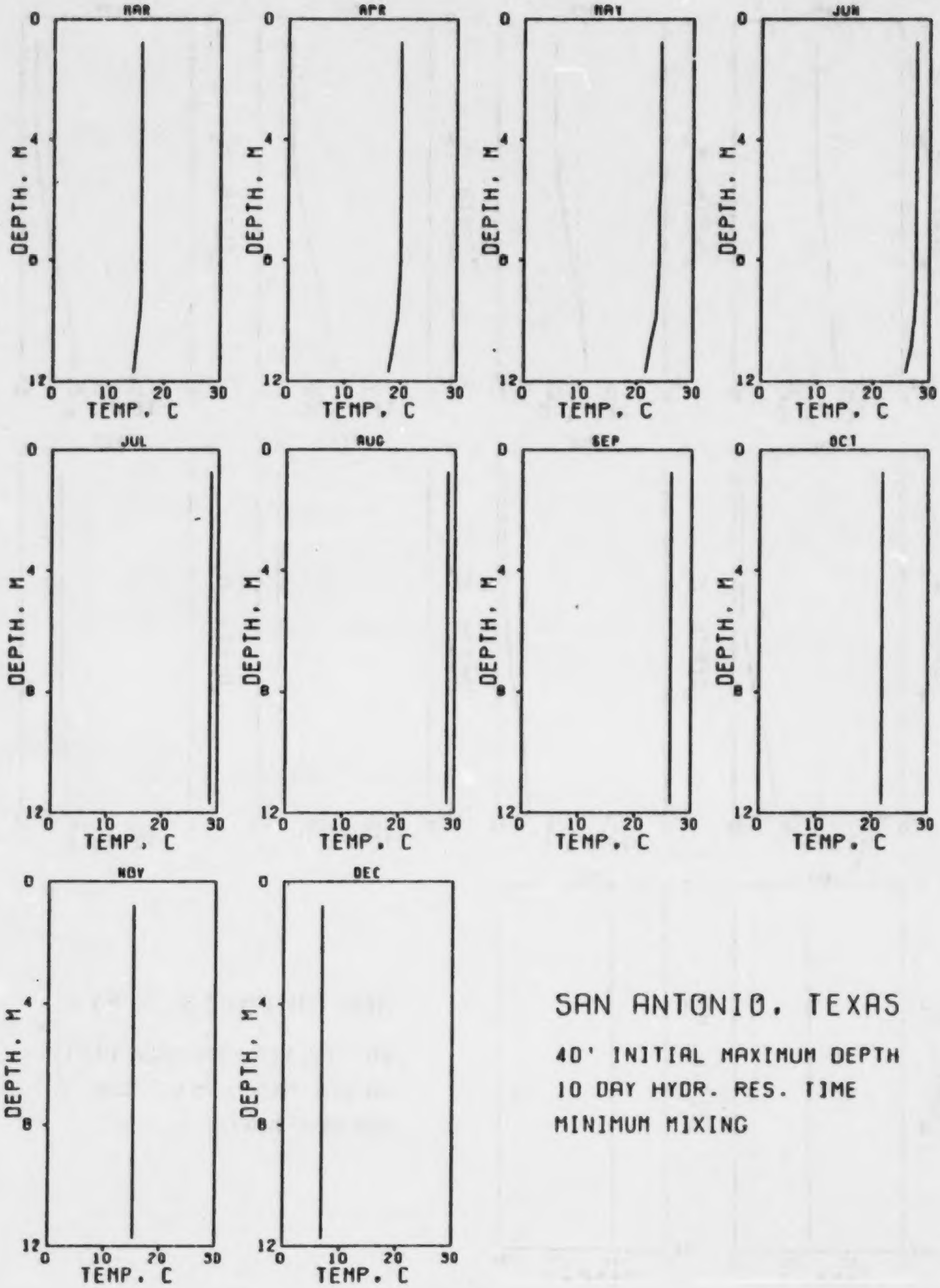


SAN ANTONIO, TEXAS
20' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



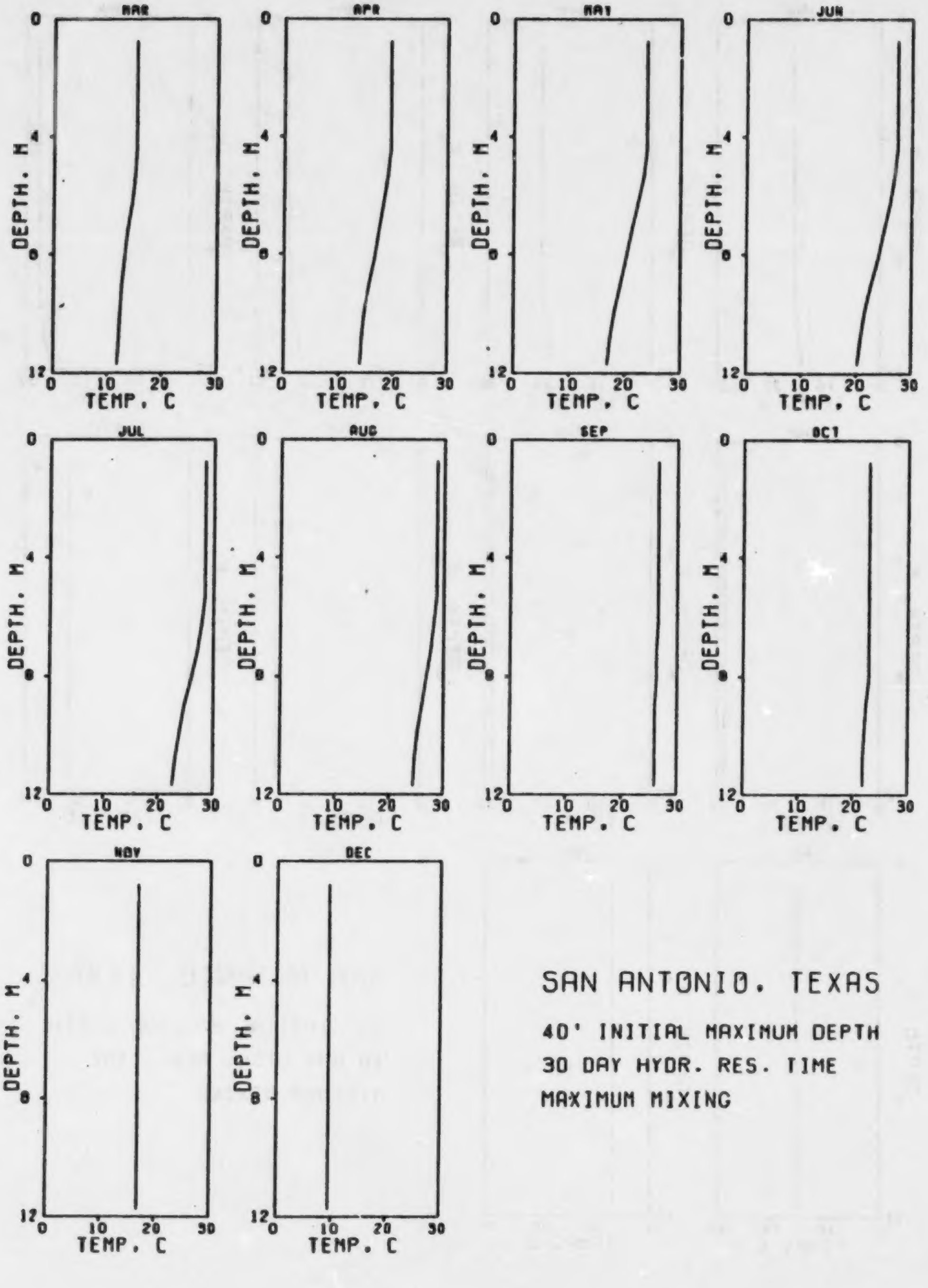
SAN ANTONIO, TEXAS

40' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



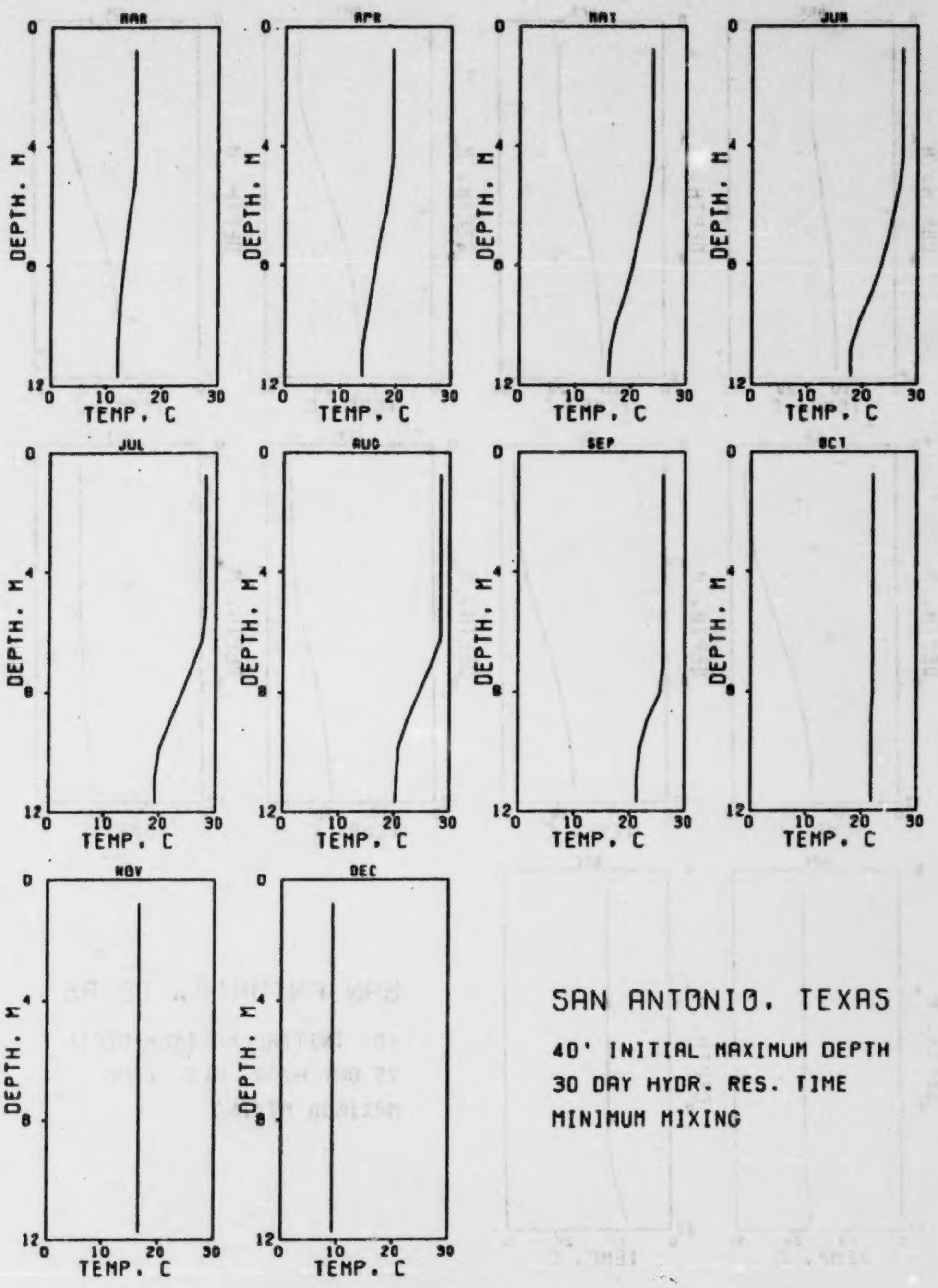
SAN ANTONIO, TEXAS

40' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



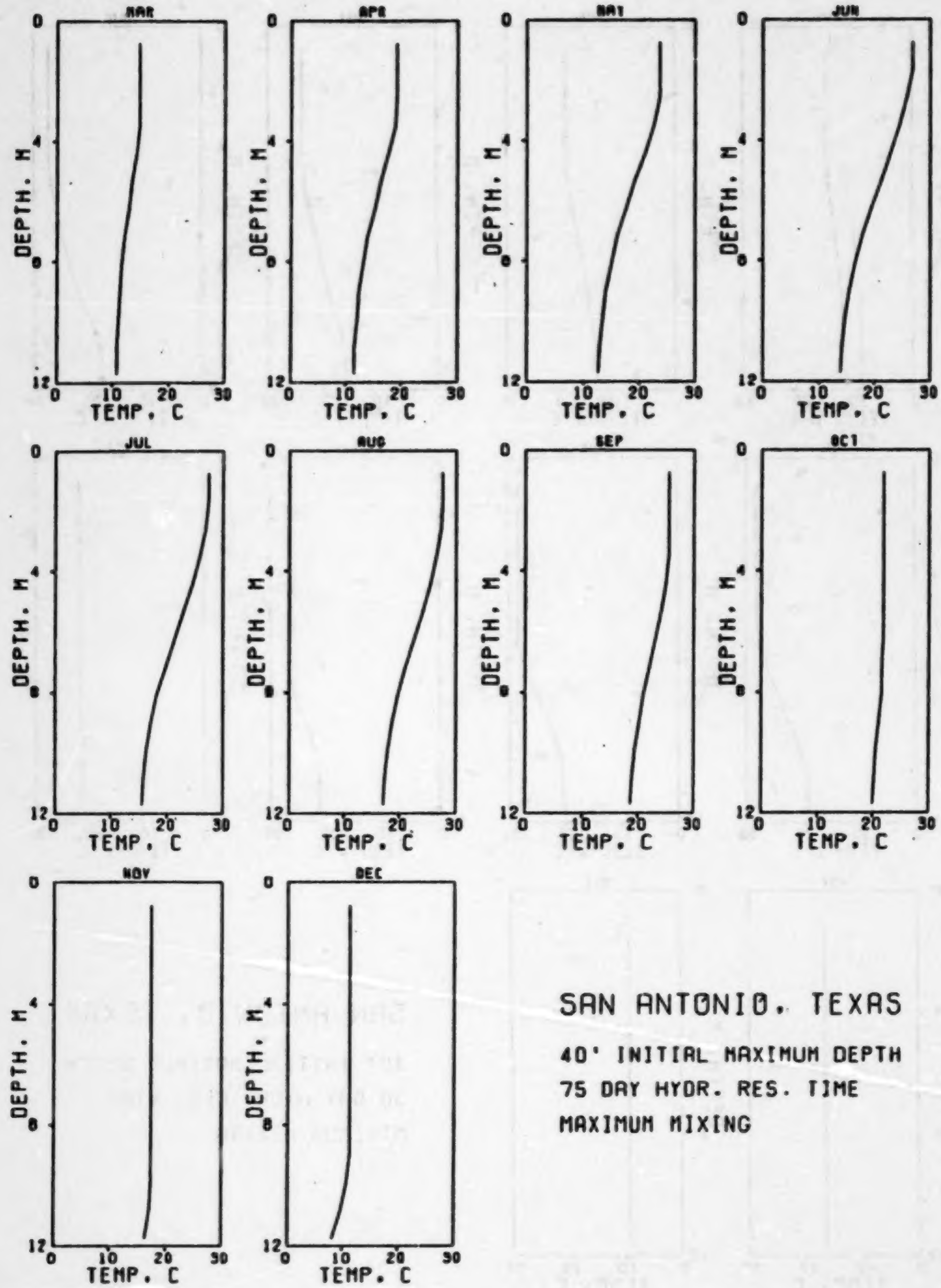
SAN ANTONIO, TEXAS

40' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



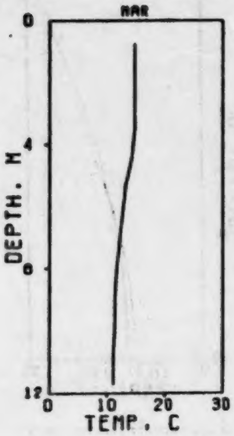
SAN ANTONIO. TEXAS

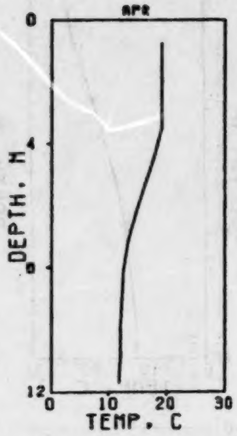
40' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING

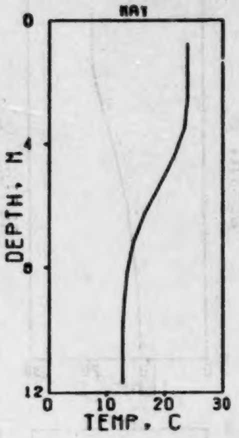


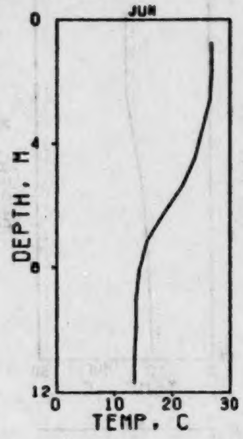
SAN ANTONIO, TEXAS

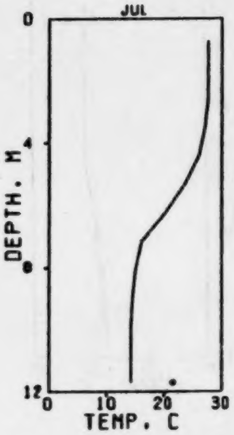
40' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING

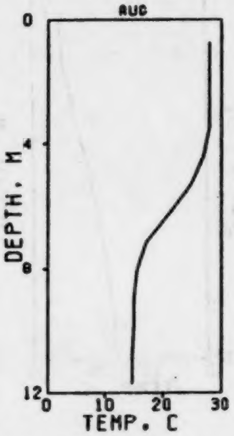


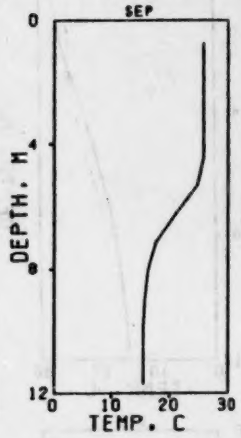


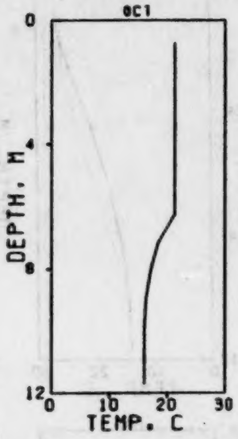


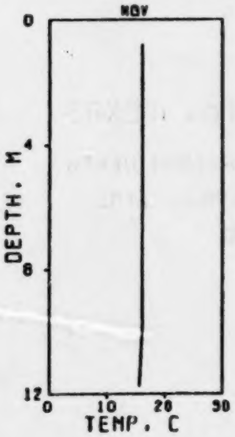


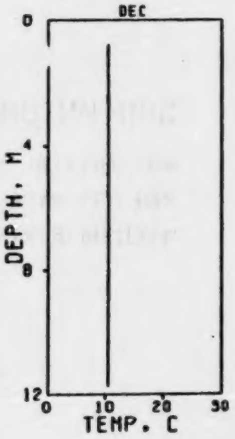






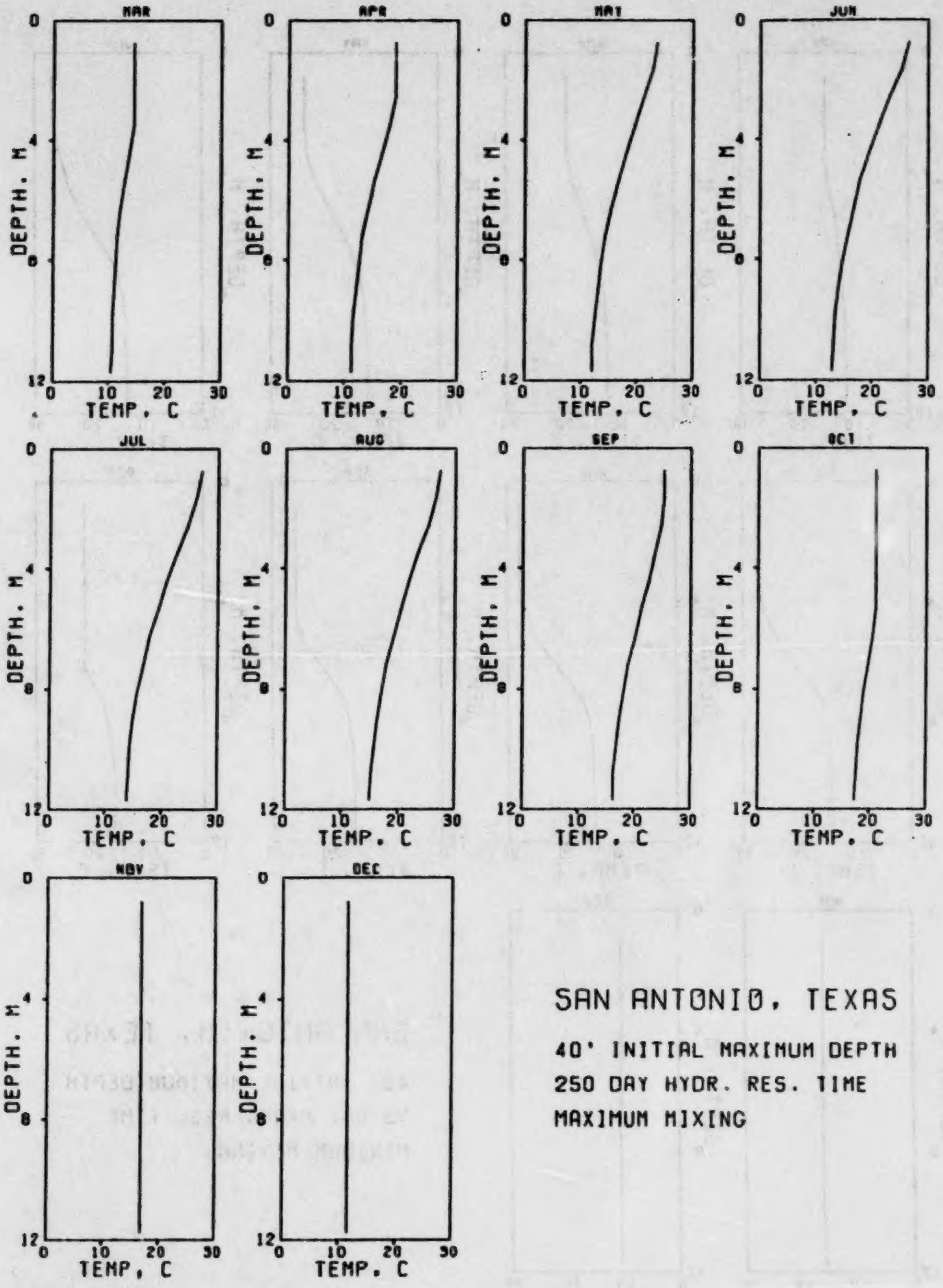






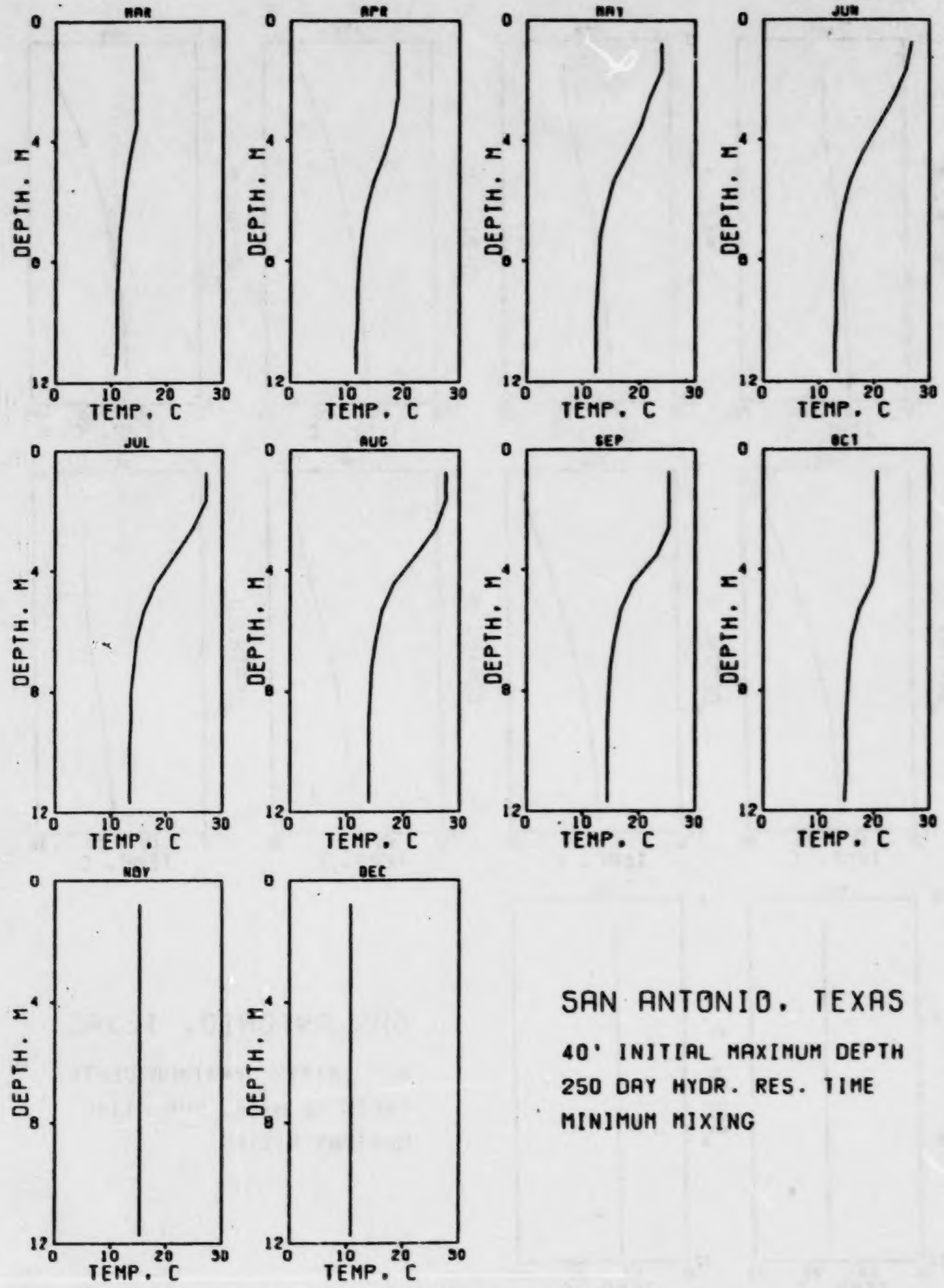
SAN ANTONIO, TEXAS

40' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



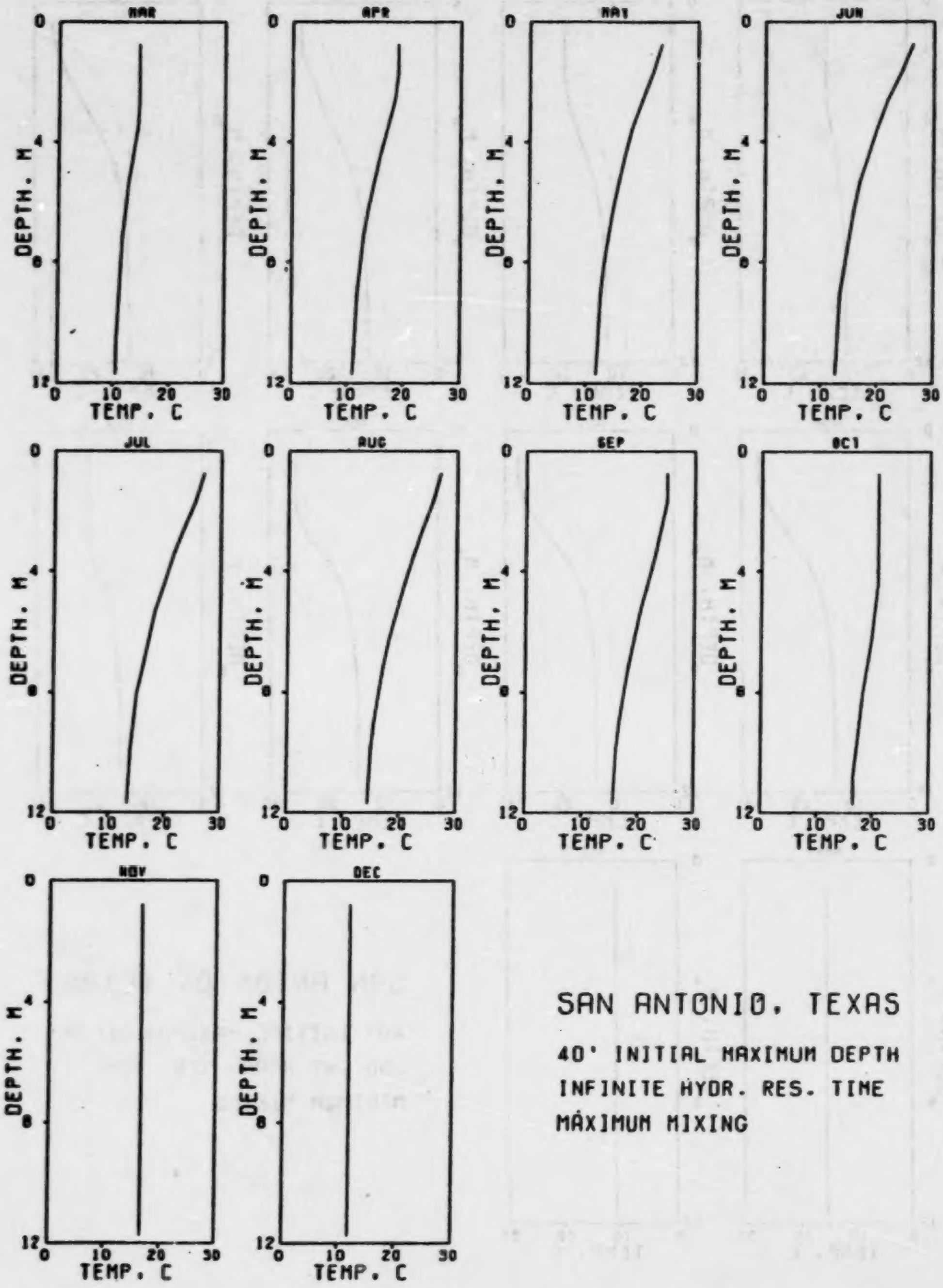
SAN ANTONIO, TEXAS

40' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



SAN ANTONIO, TEXAS

40' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING

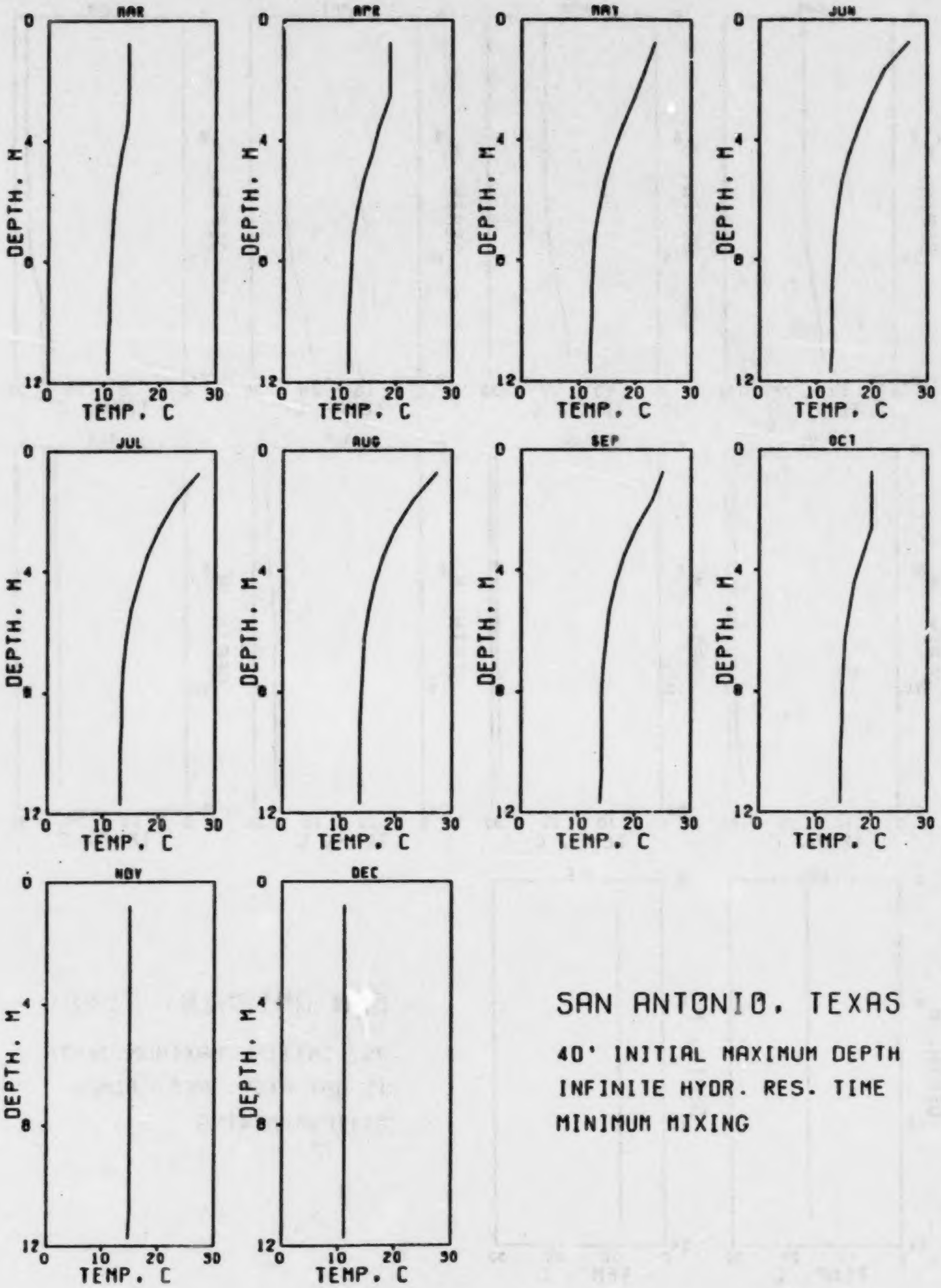


SAN ANTONIO, TEXAS

40' INITIAL MAXIMUM DEPTH

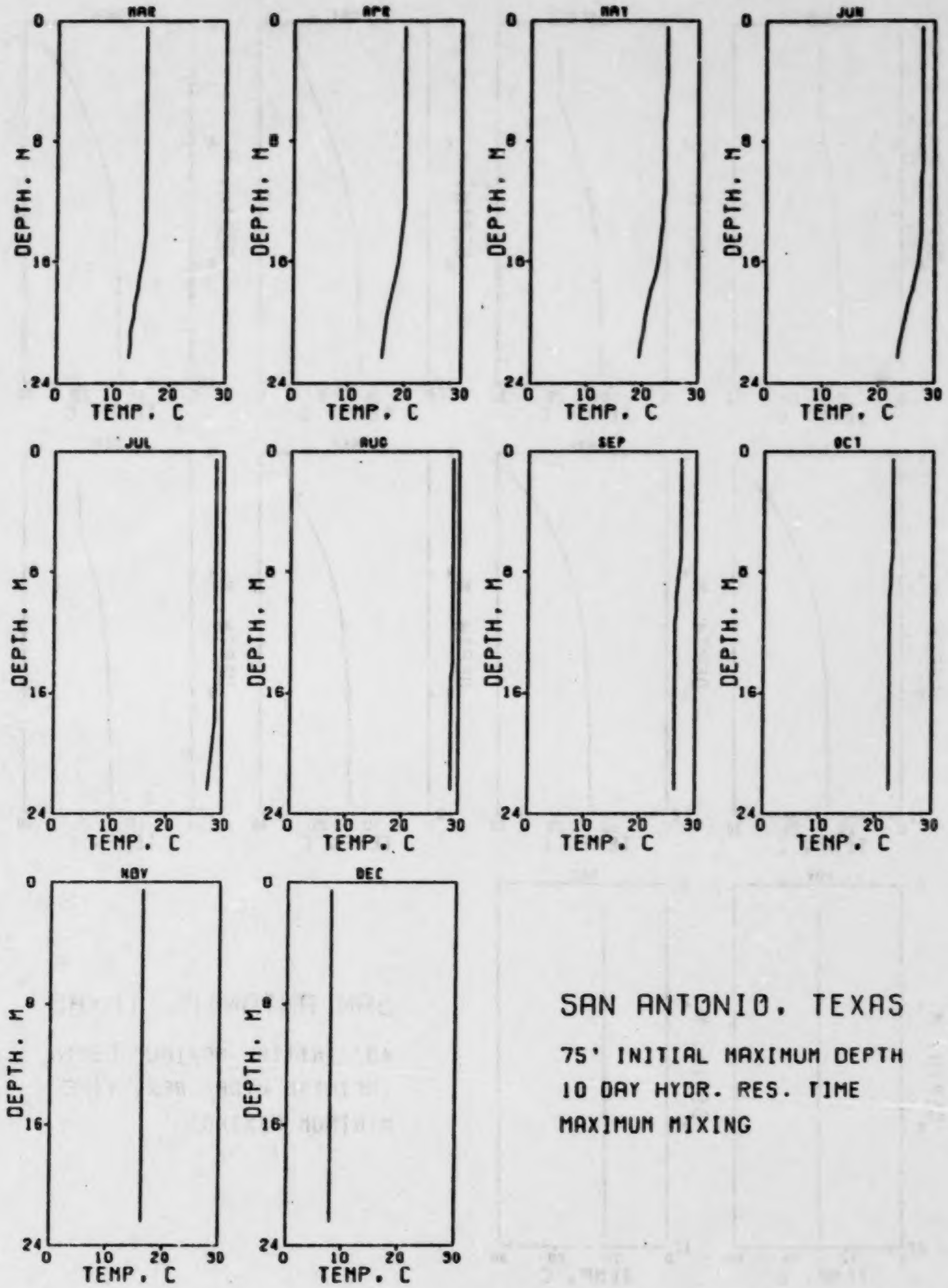
INFINITE HYDR. RES. TIME

MAXIMUM MIXING



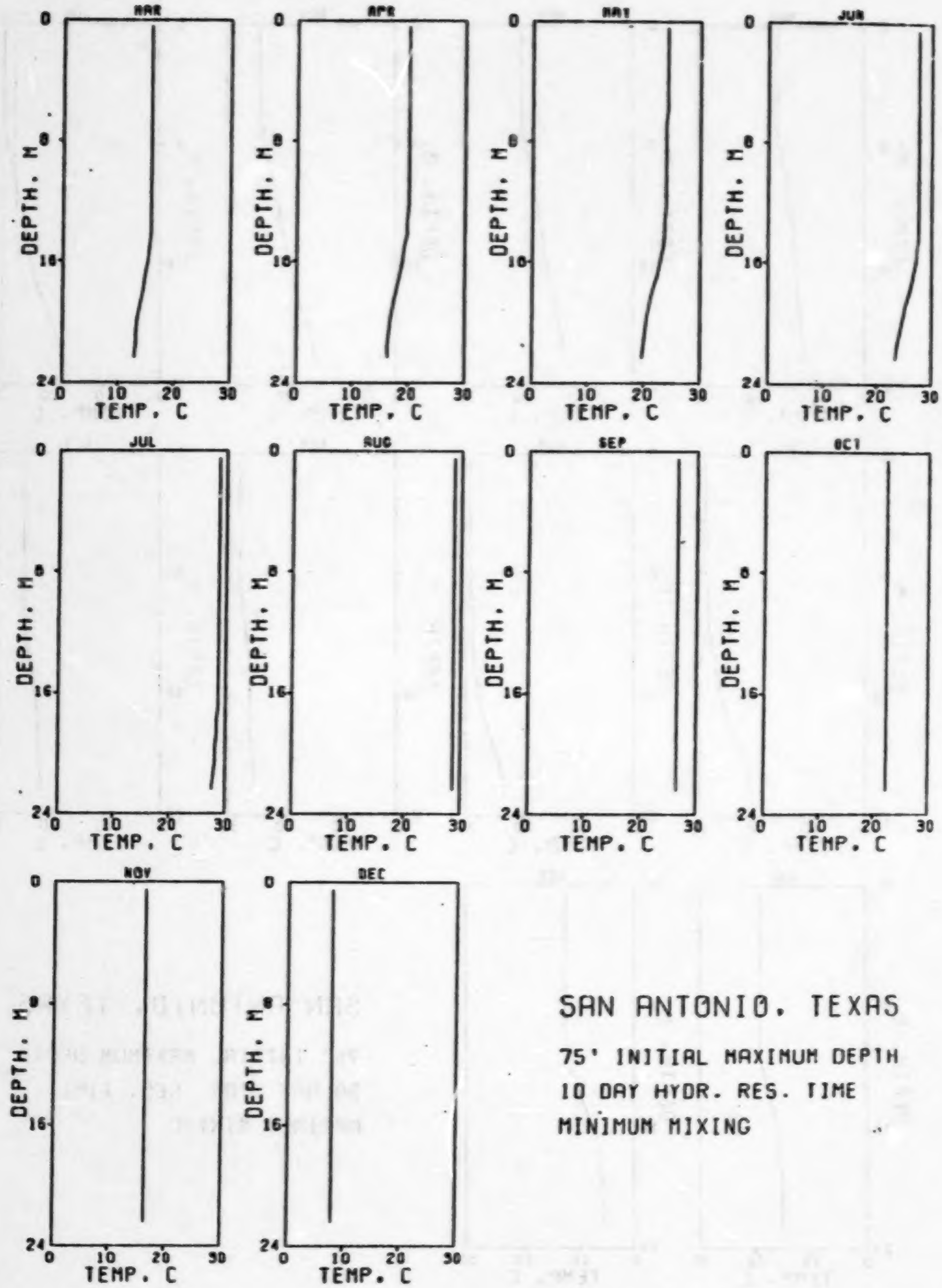
SAN ANTONIO, TEXAS

40' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



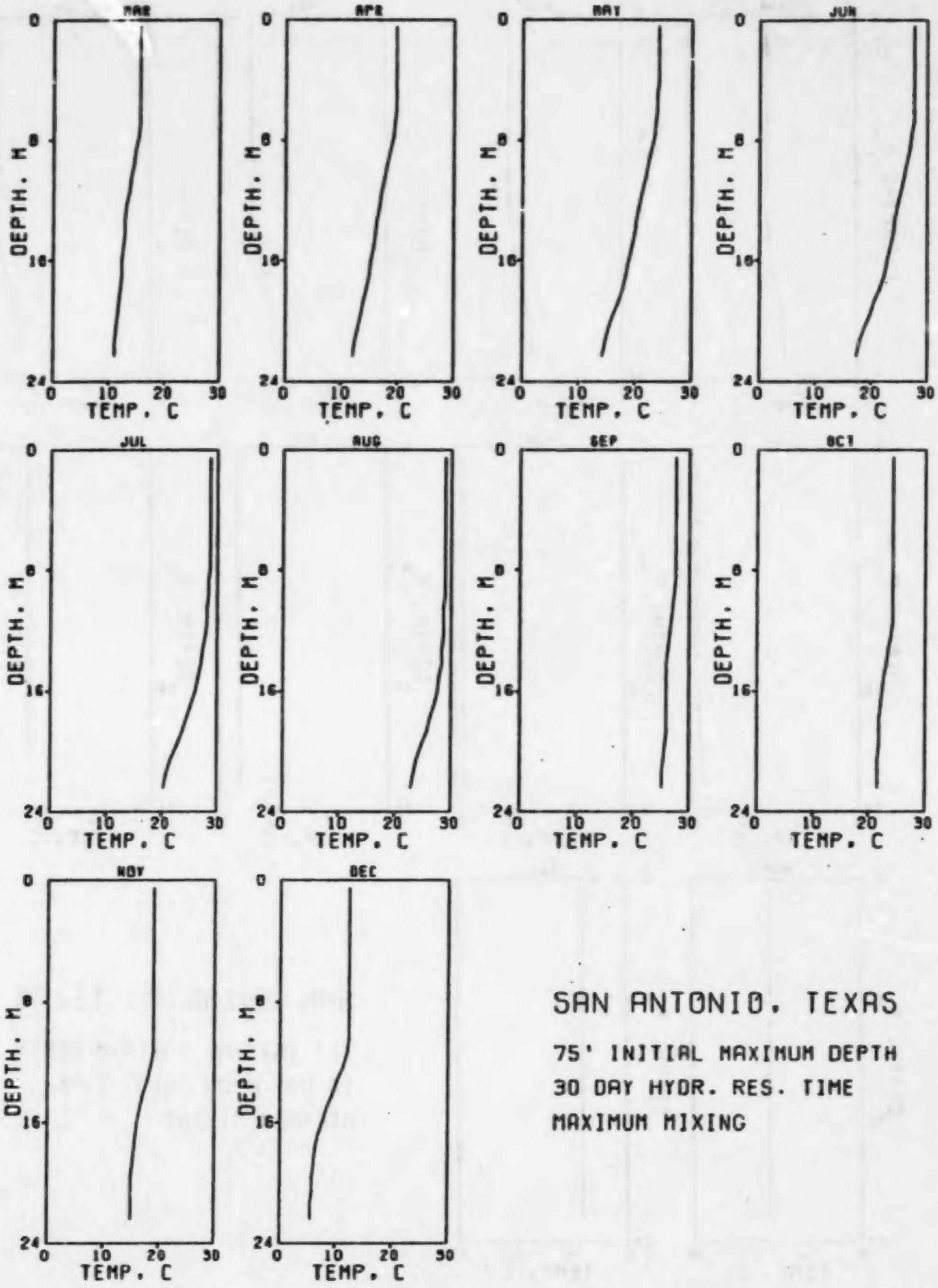
SAN ANTONIO, TEXAS

75' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



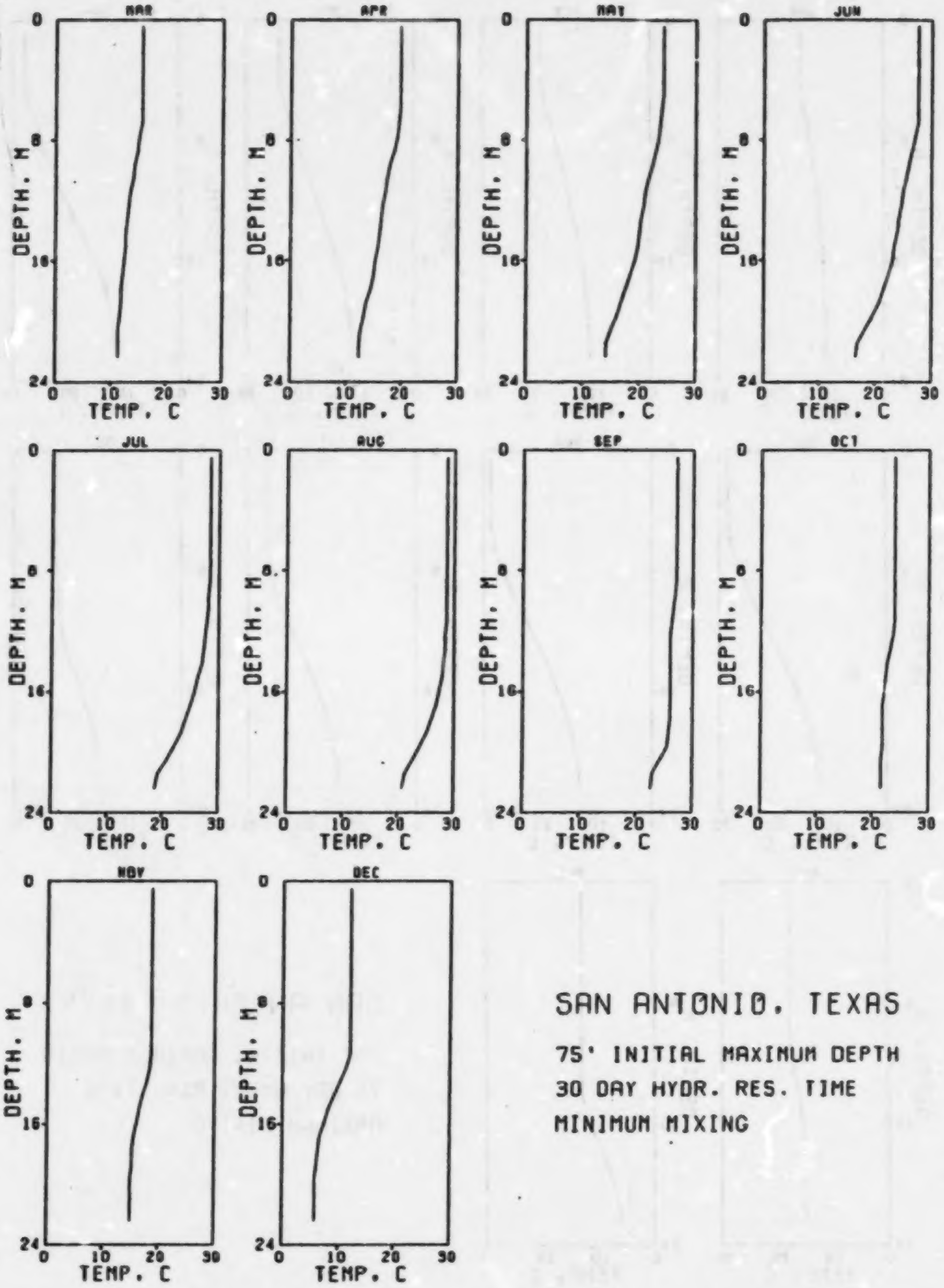
SAN ANTONIO, TEXAS

75' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



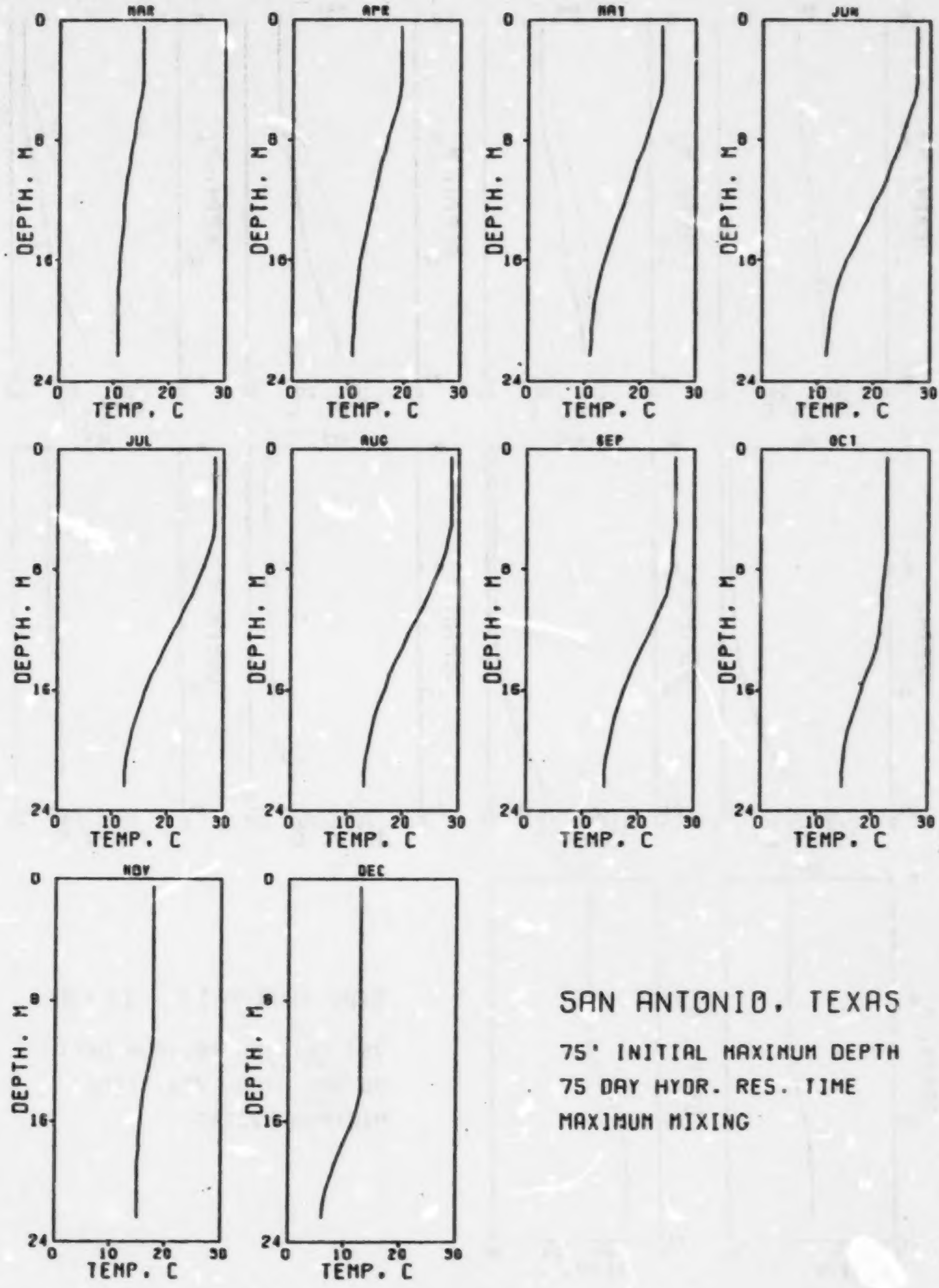
SAN ANTONIO, TEXAS

75' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



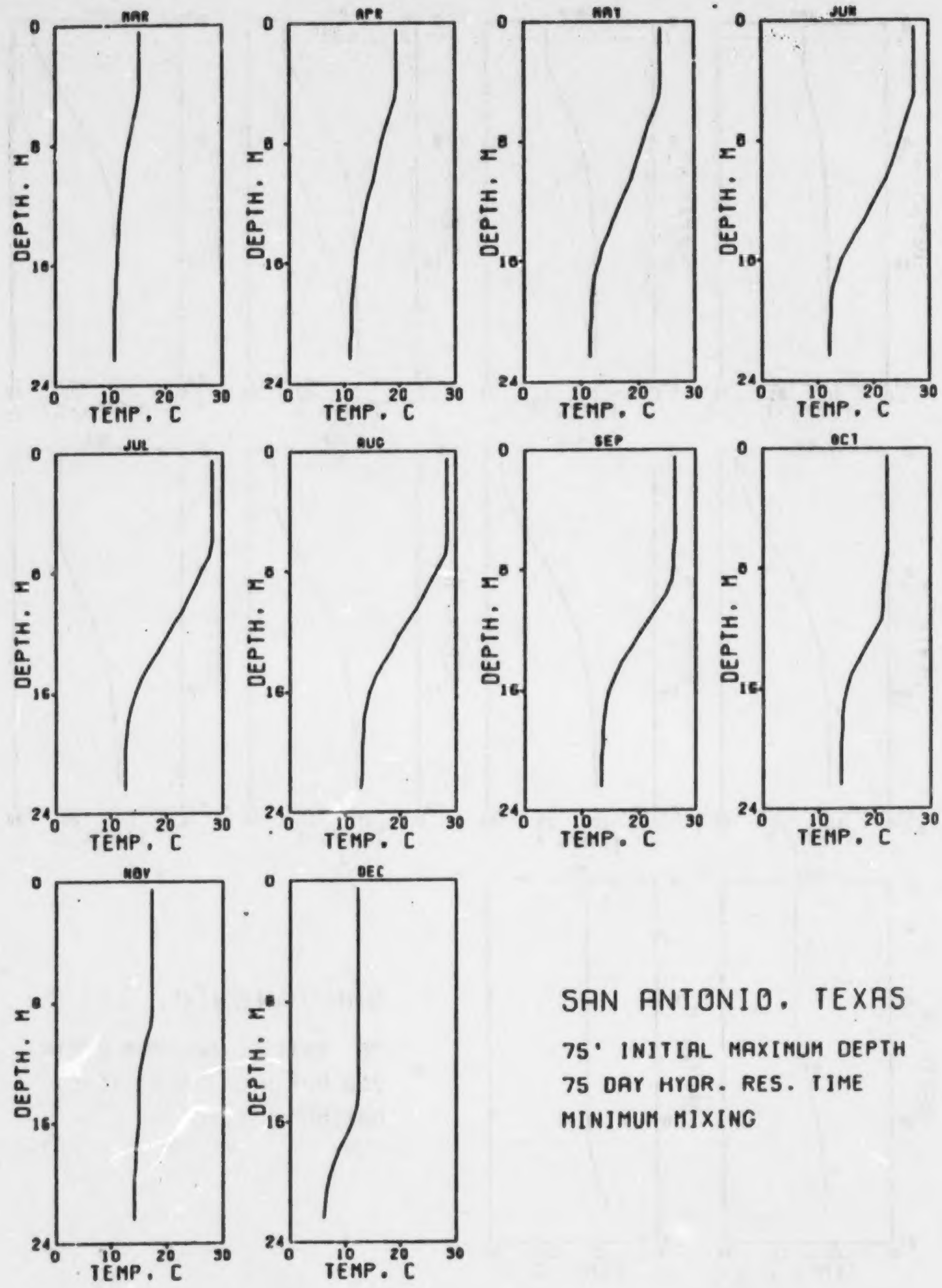
SAN ANTONIO, TEXAS

75' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



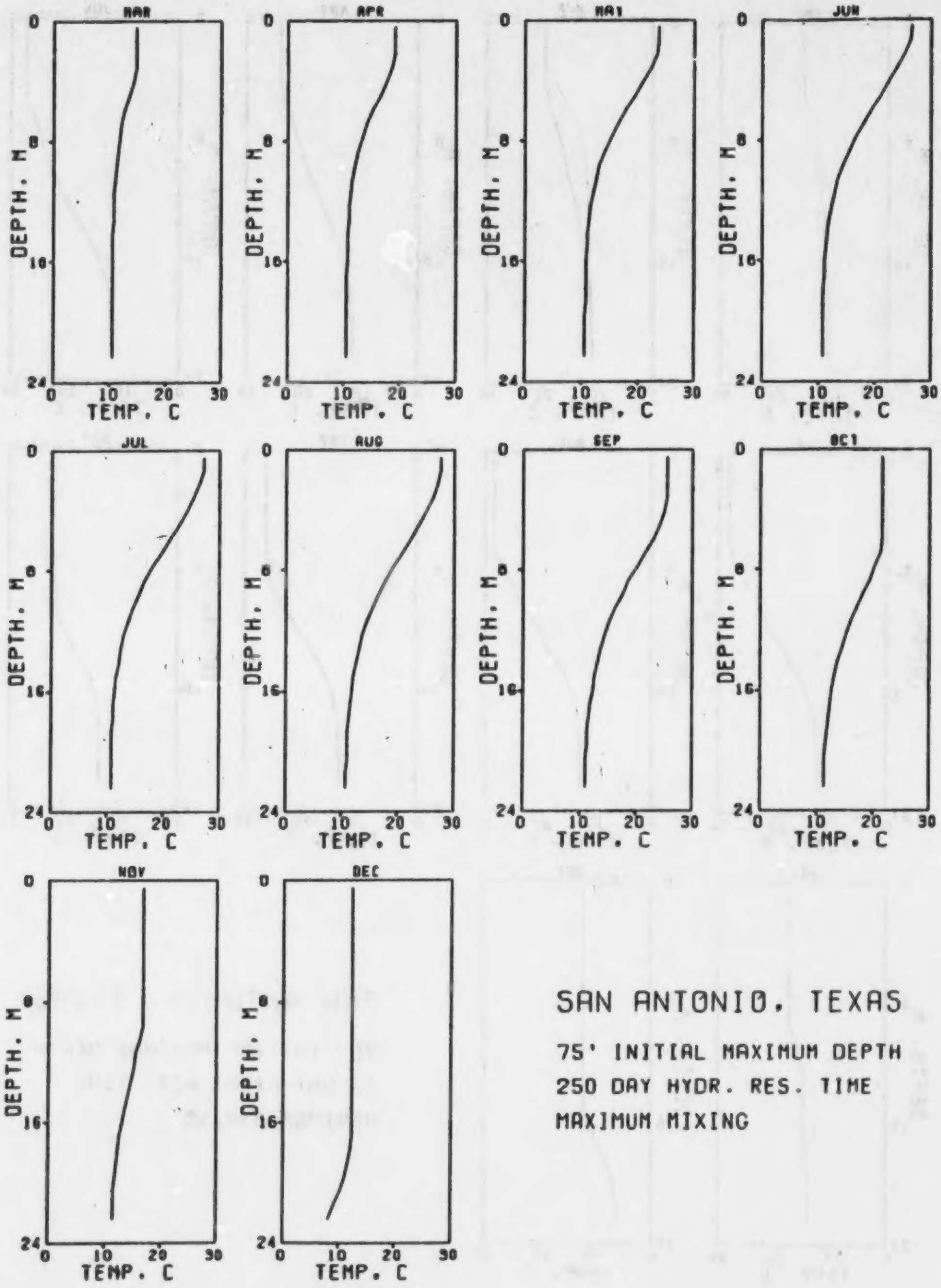
SAN ANTONIO, TEXAS

75' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



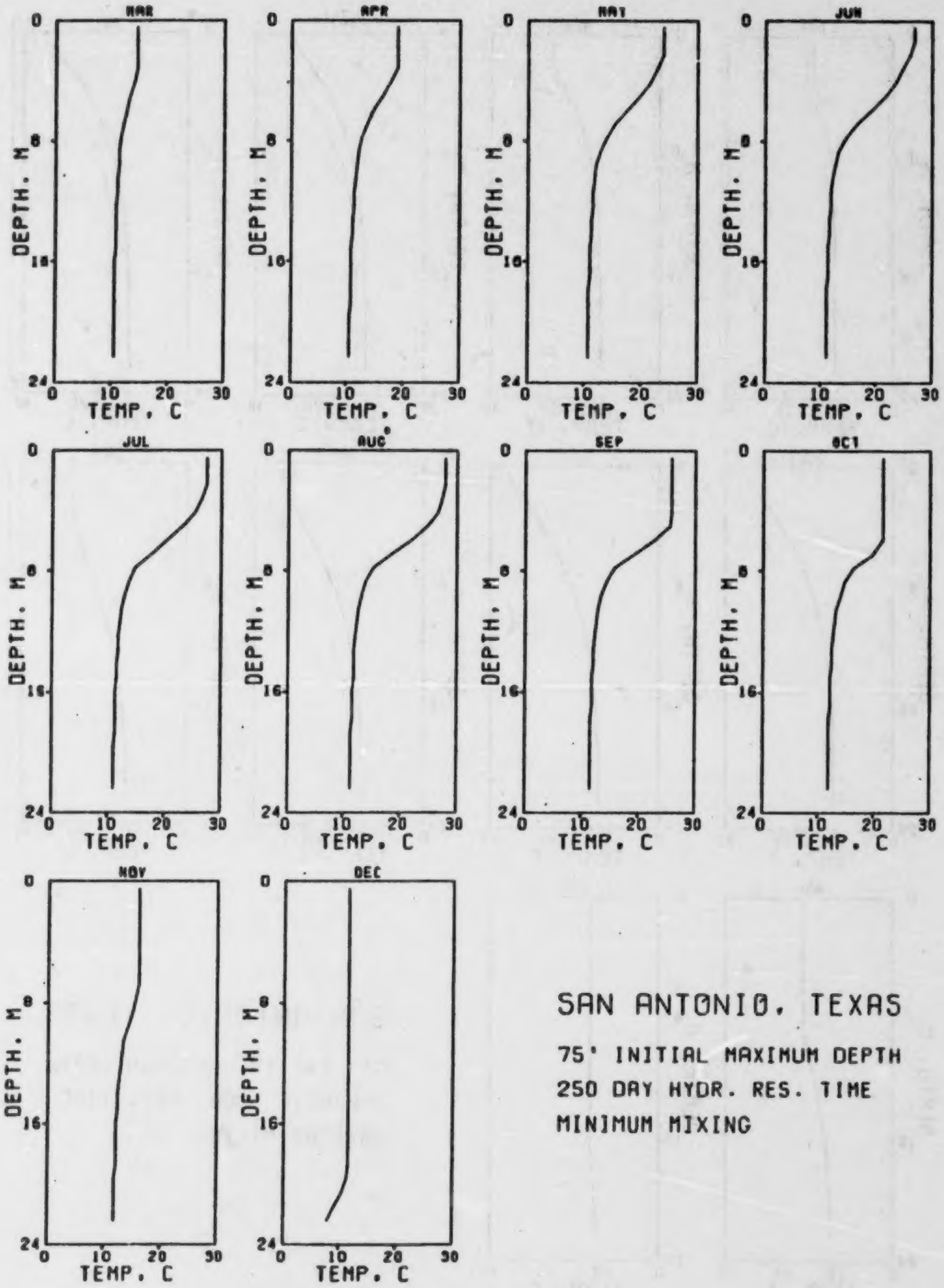
SAN ANTONIO, TEXAS

75' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



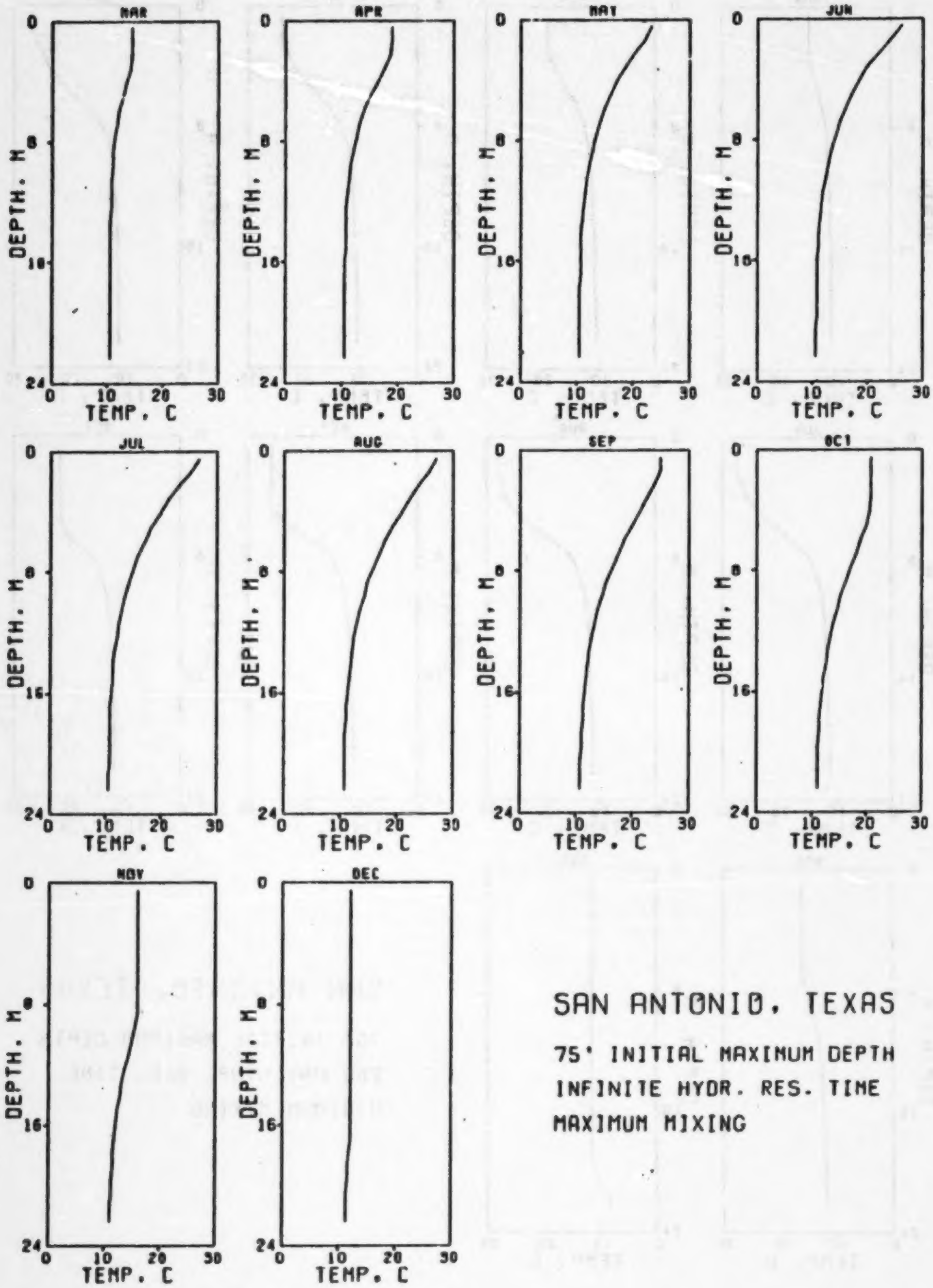
SAN ANTONIO, TEXAS

75' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



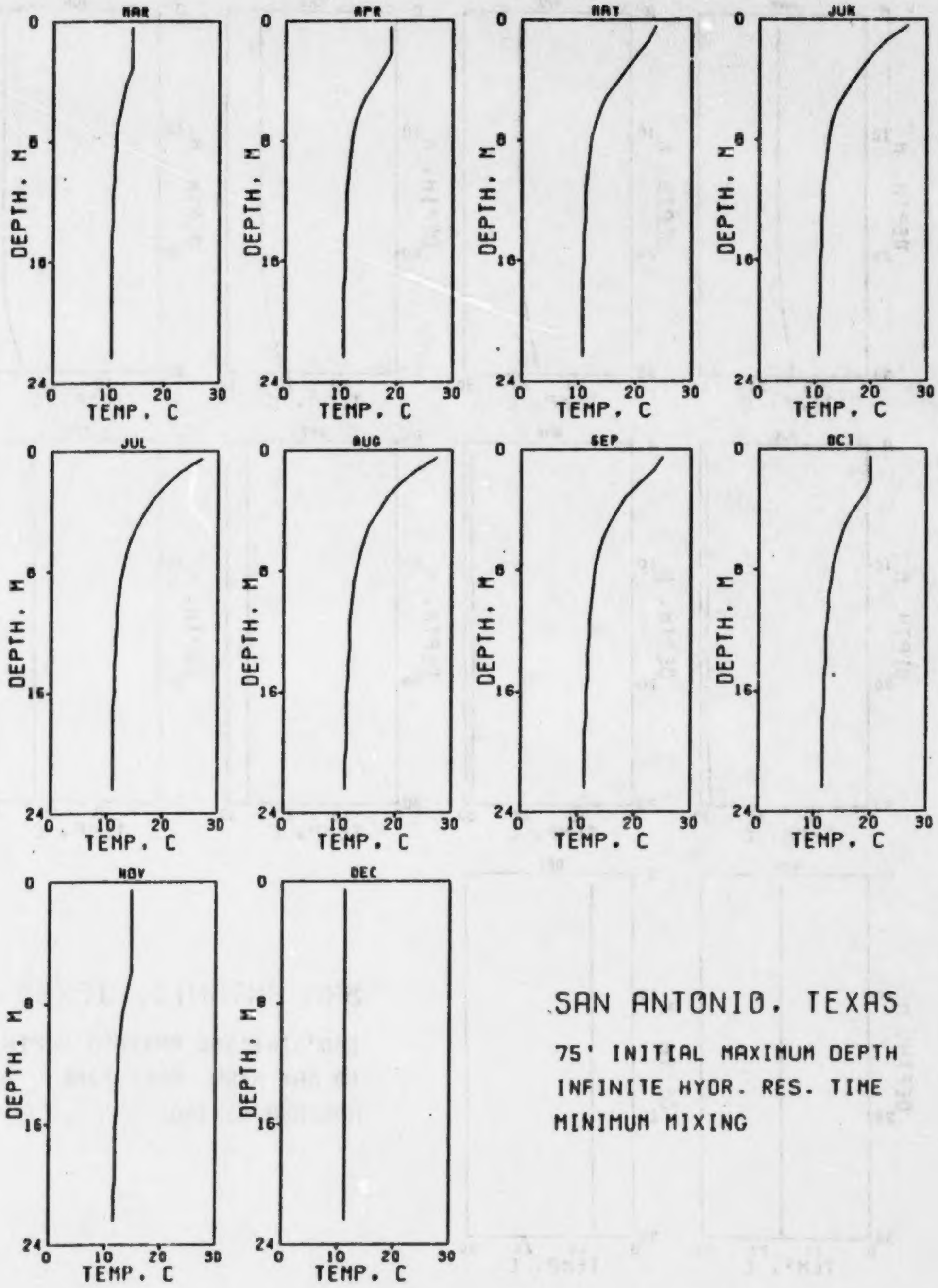
SAN ANTONIO, TEXAS

75' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



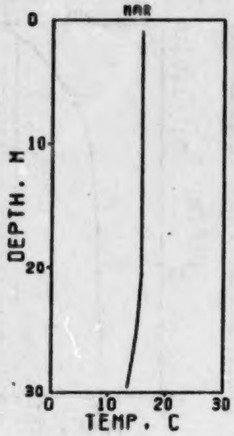
SAN ANTONIO, TEXAS

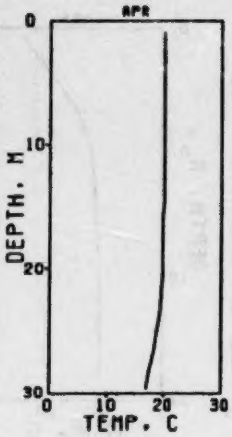
75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING

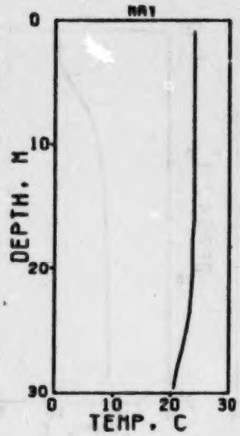


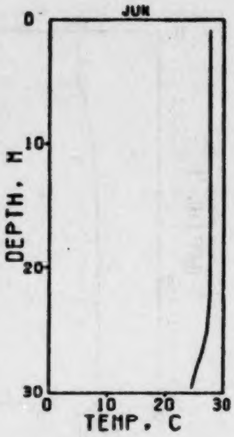
SAN ANTONIO, TEXAS

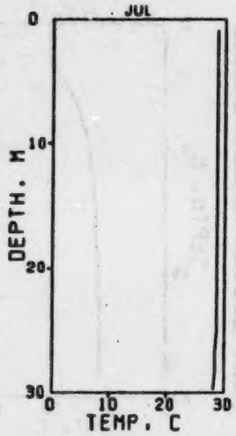
75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING

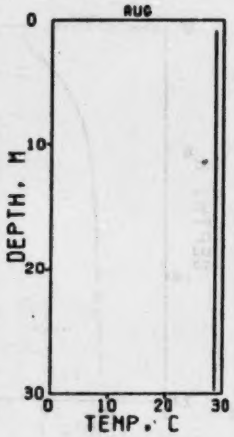


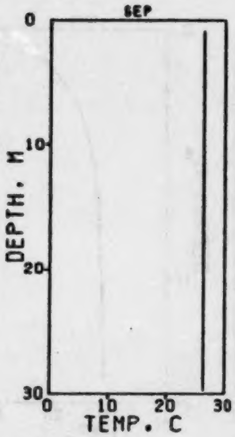


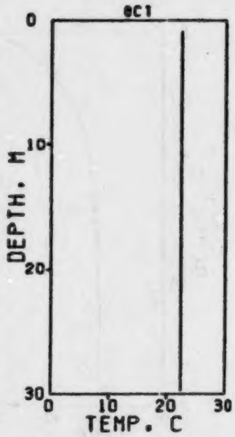


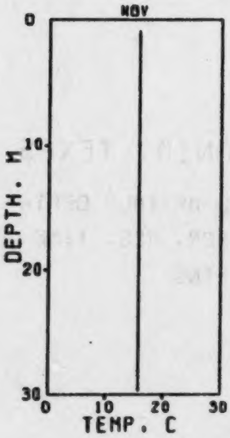


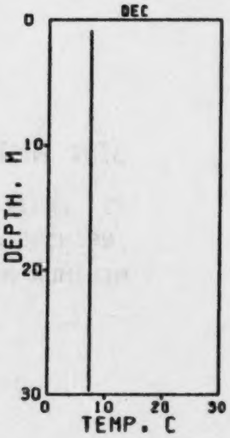






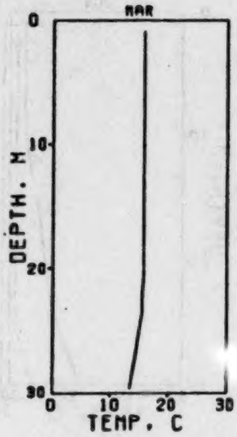


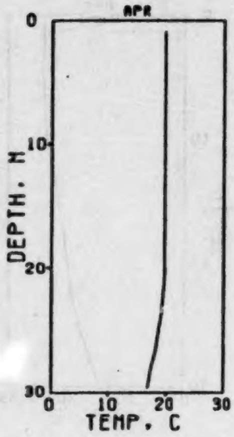


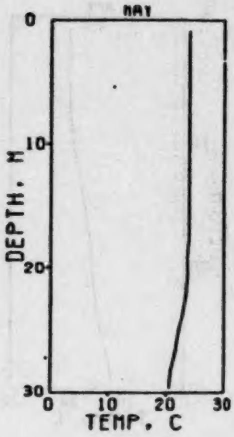


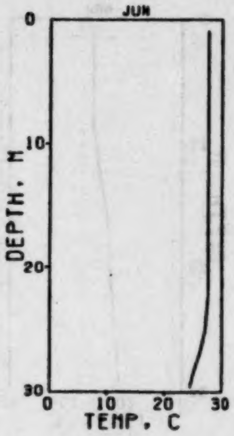
SAN ANTONIO, TEXAS

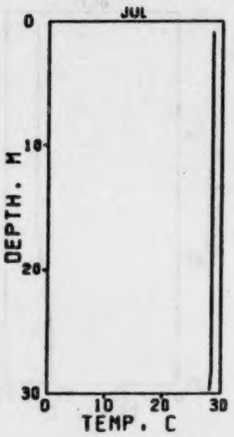
100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING

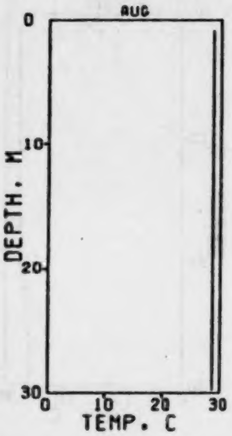


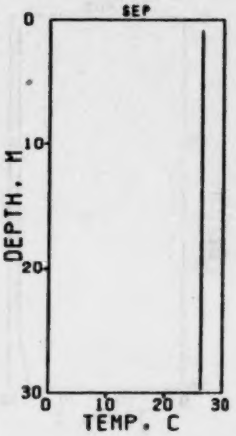


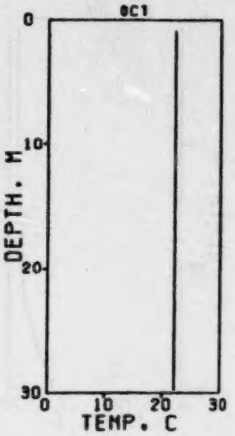


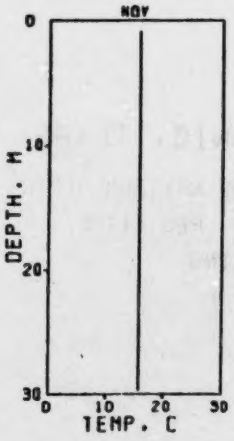


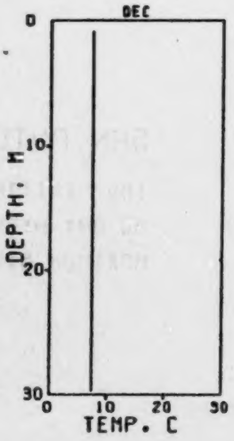






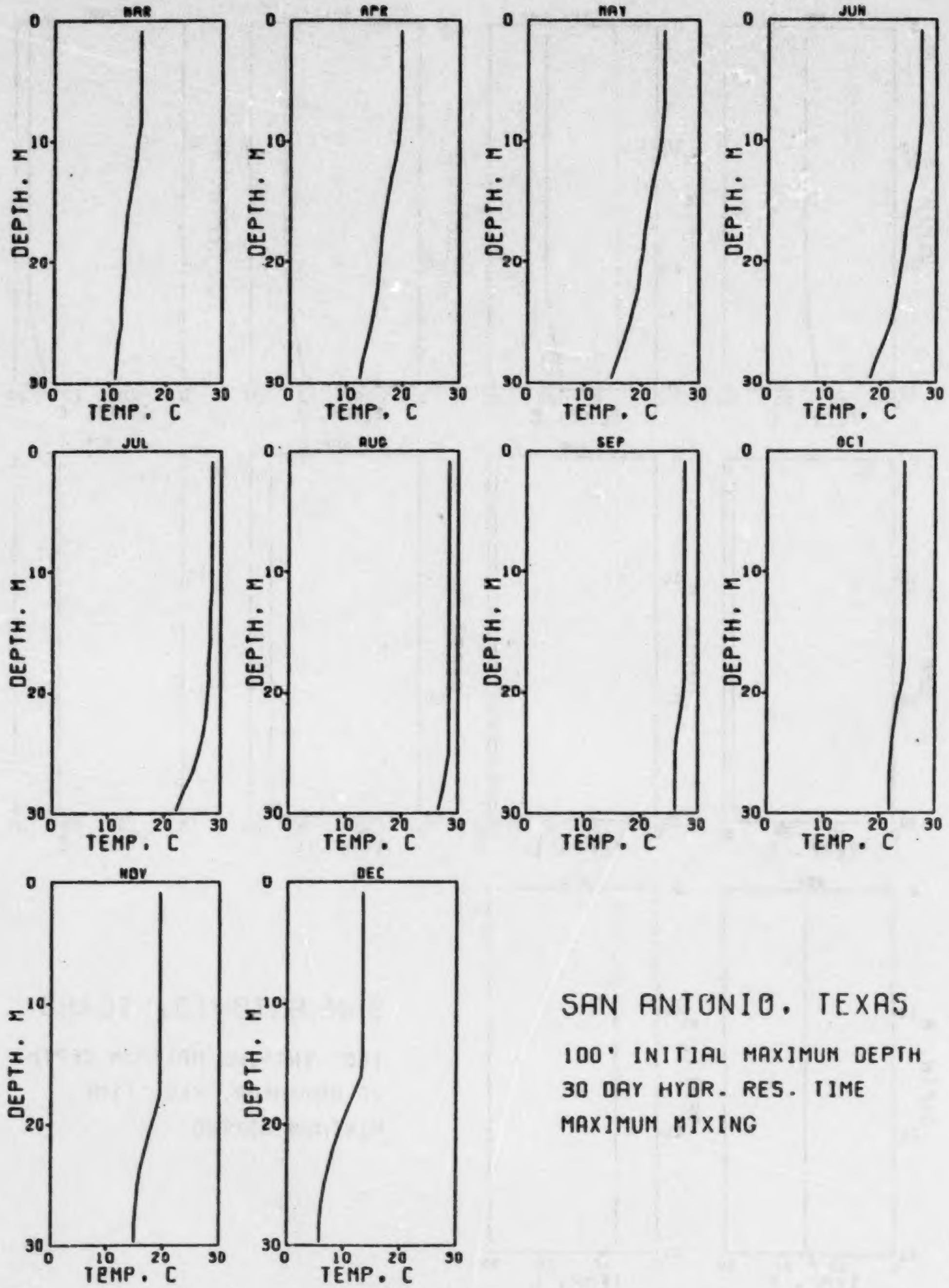






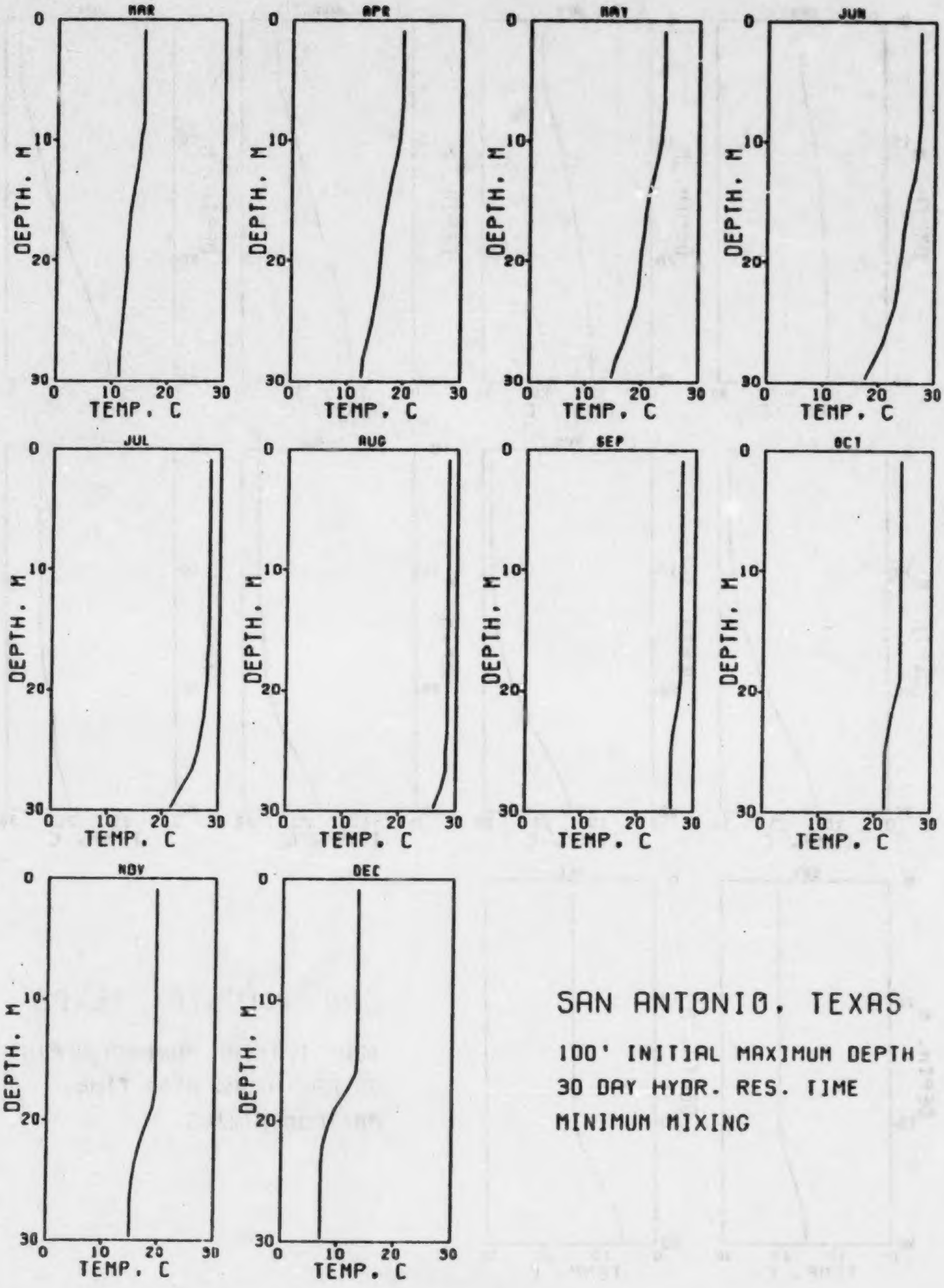
SAN ANTONIO, TEXAS

100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



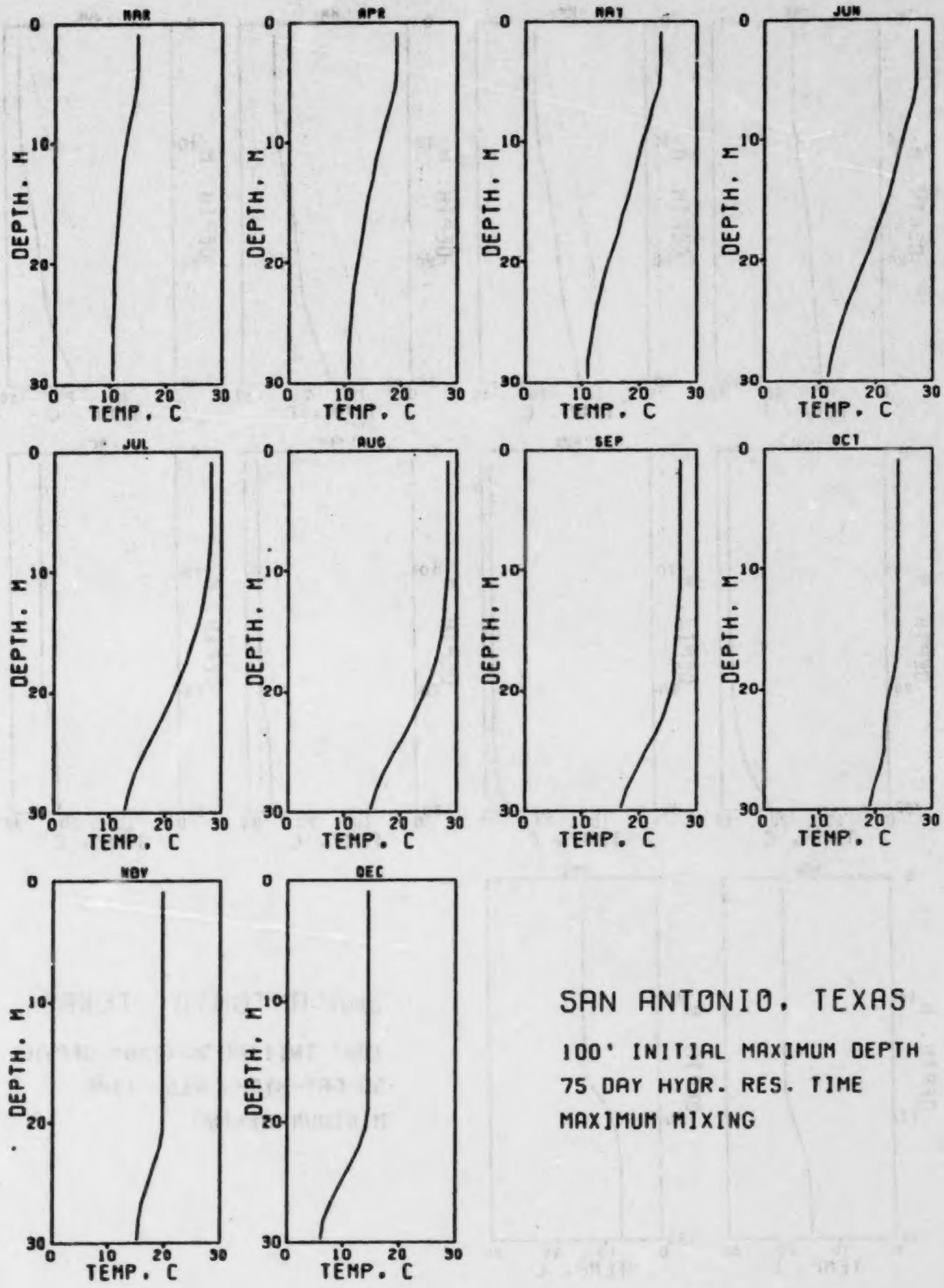
SAN ANTONIO, TEXAS

100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



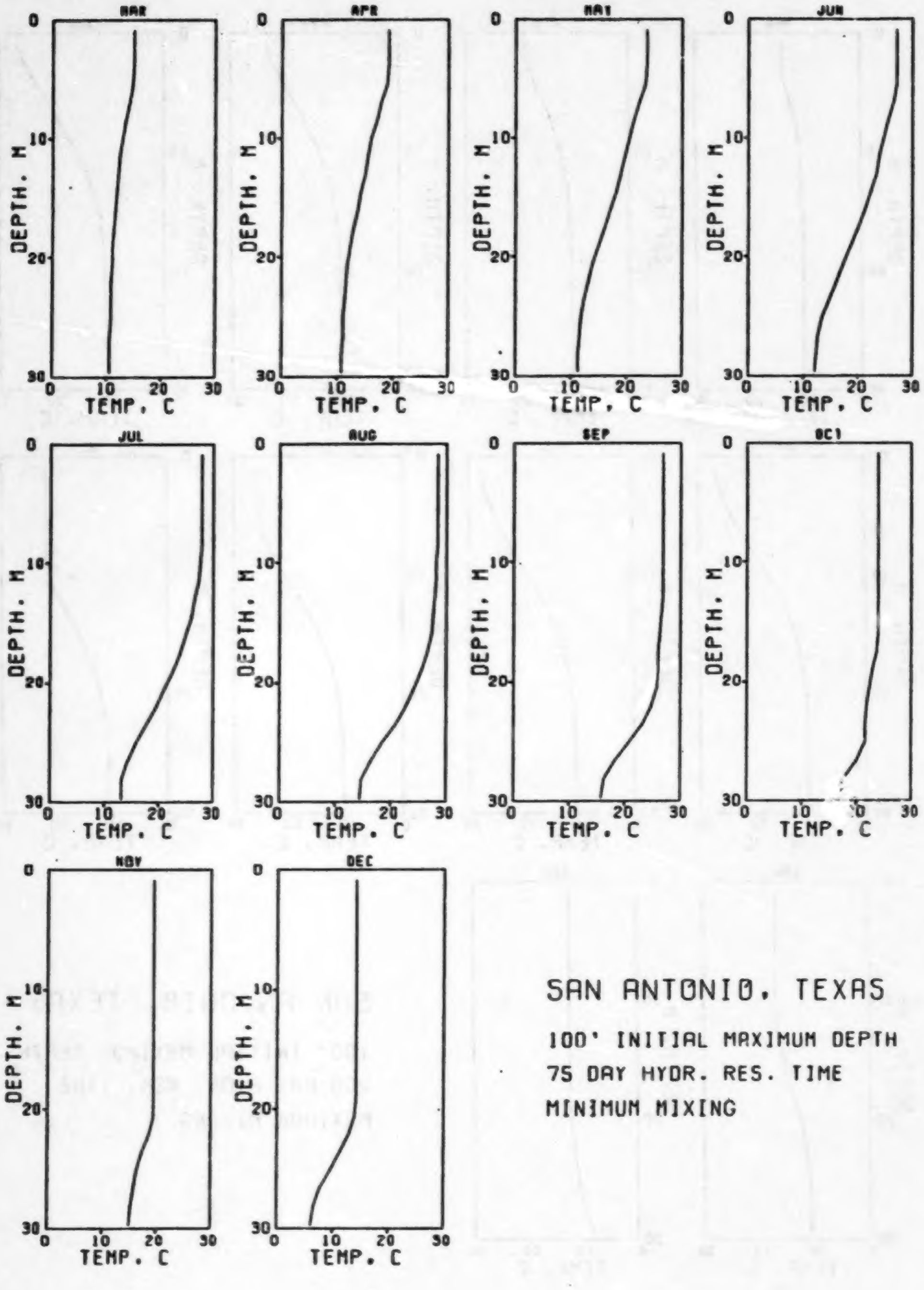
SAN ANTONIO, TEXAS

100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



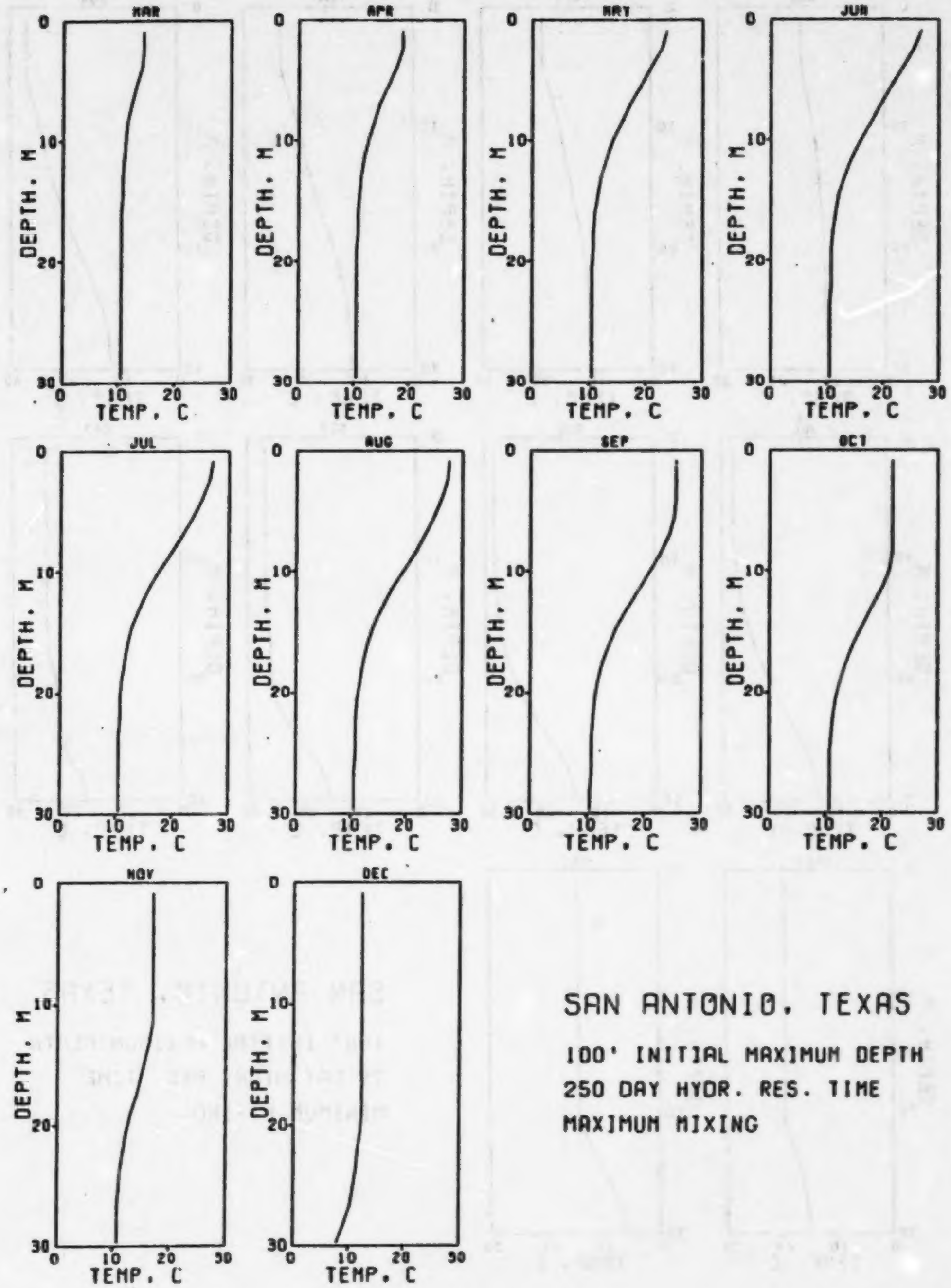
SAN ANTONIO, TEXAS

100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



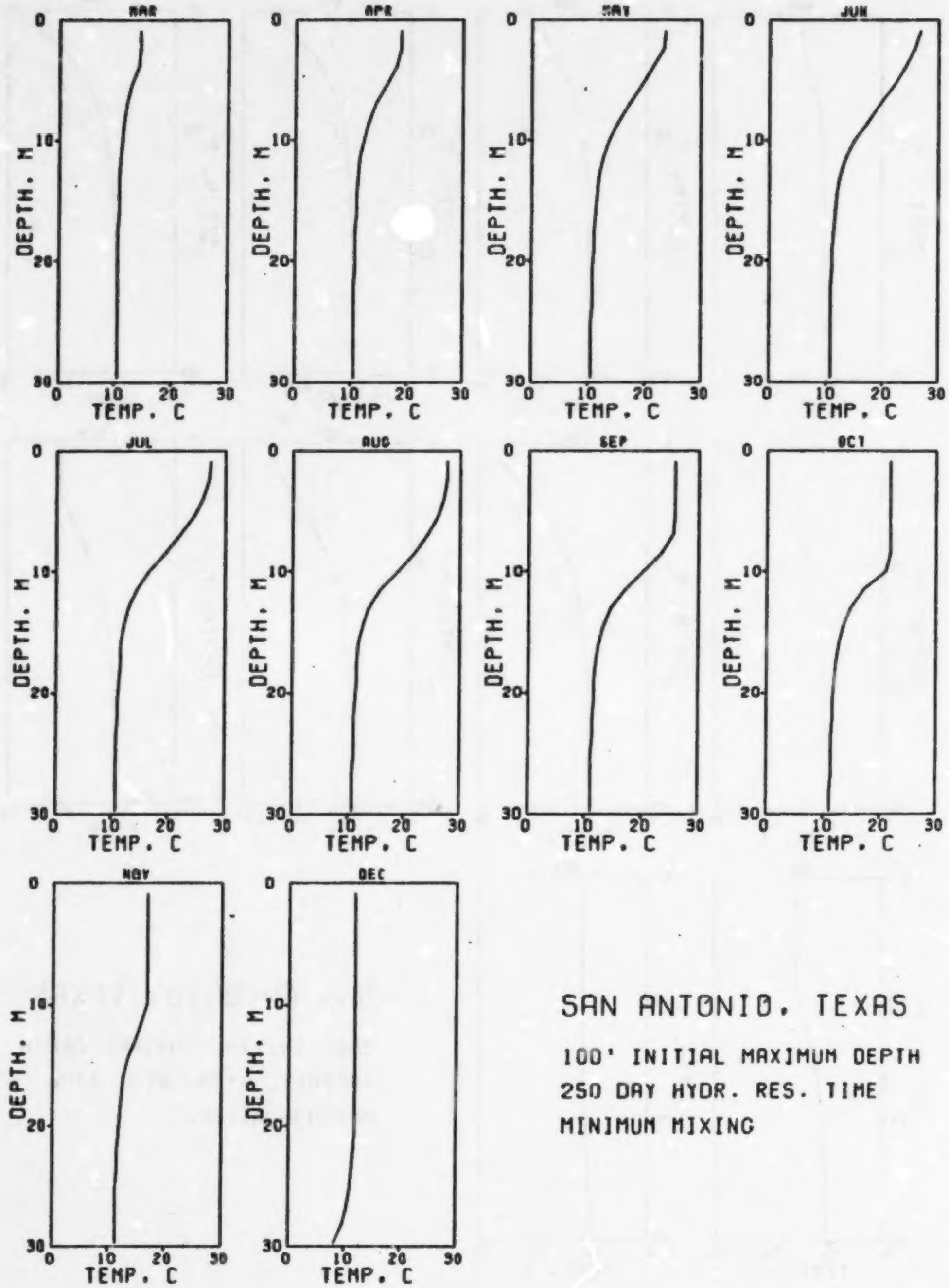
SAN ANTONIO, TEXAS

100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



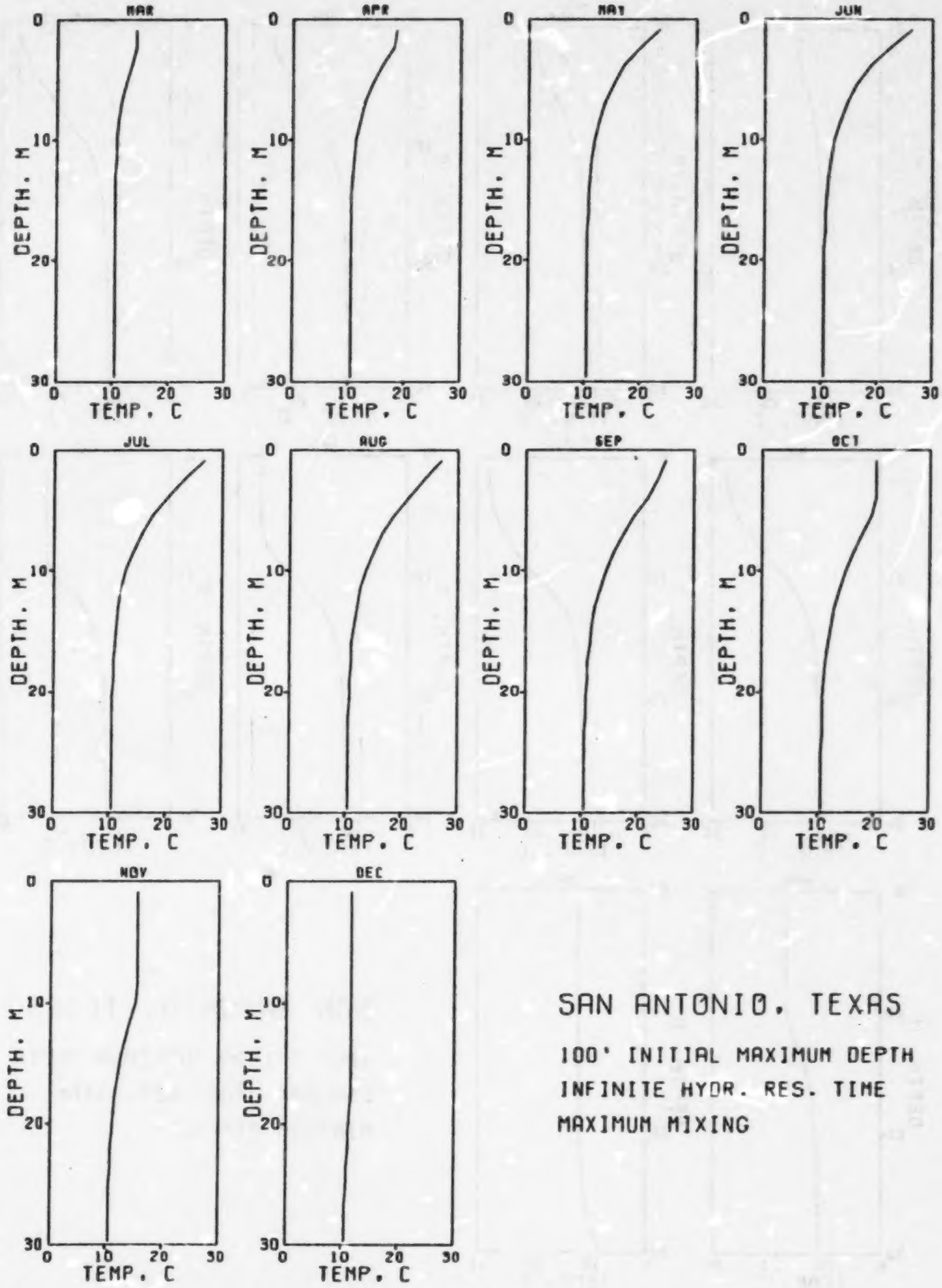
SAN ANTONIO, TEXAS

100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



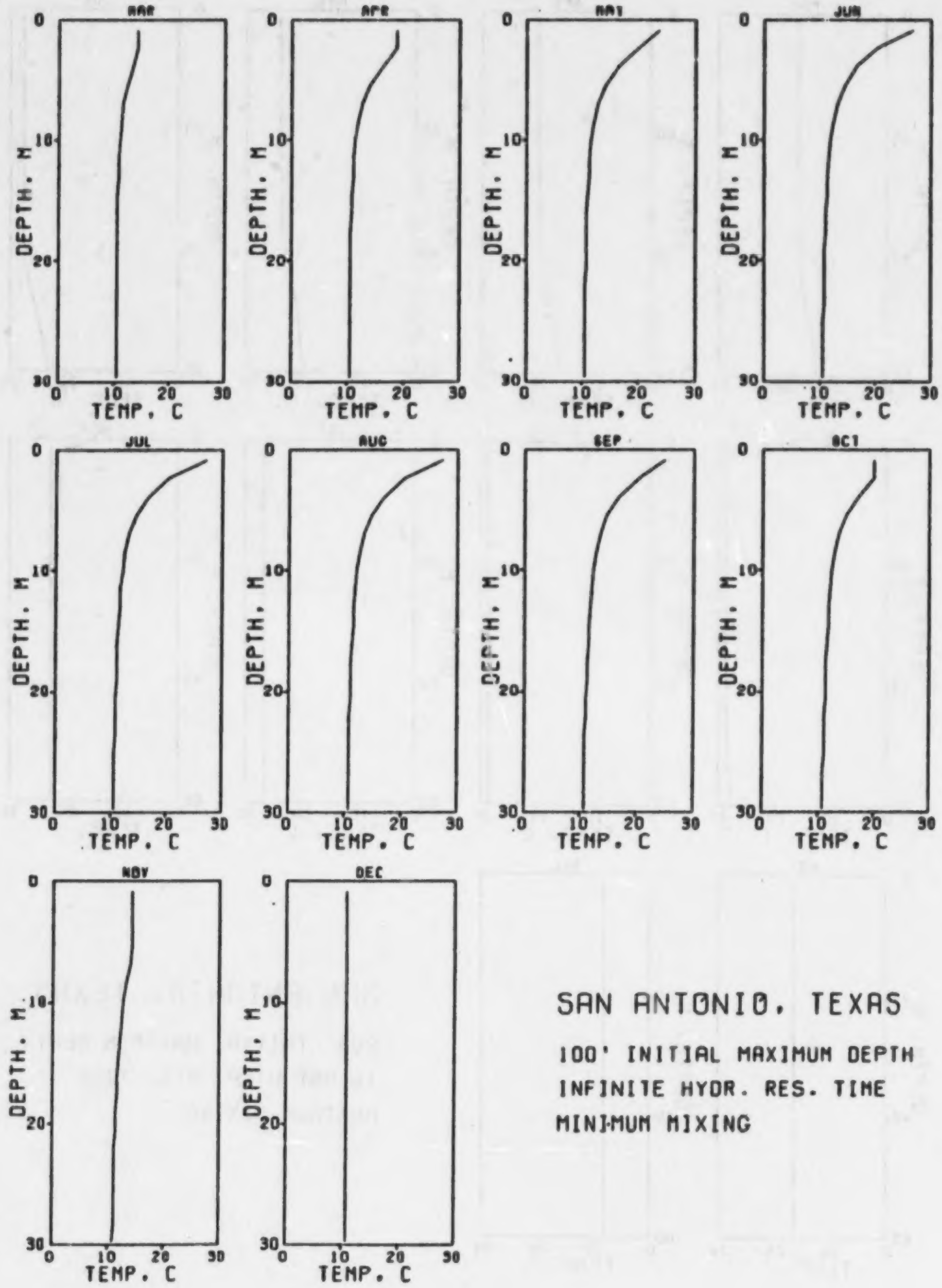
SAN ANTONIO, TEXAS

100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



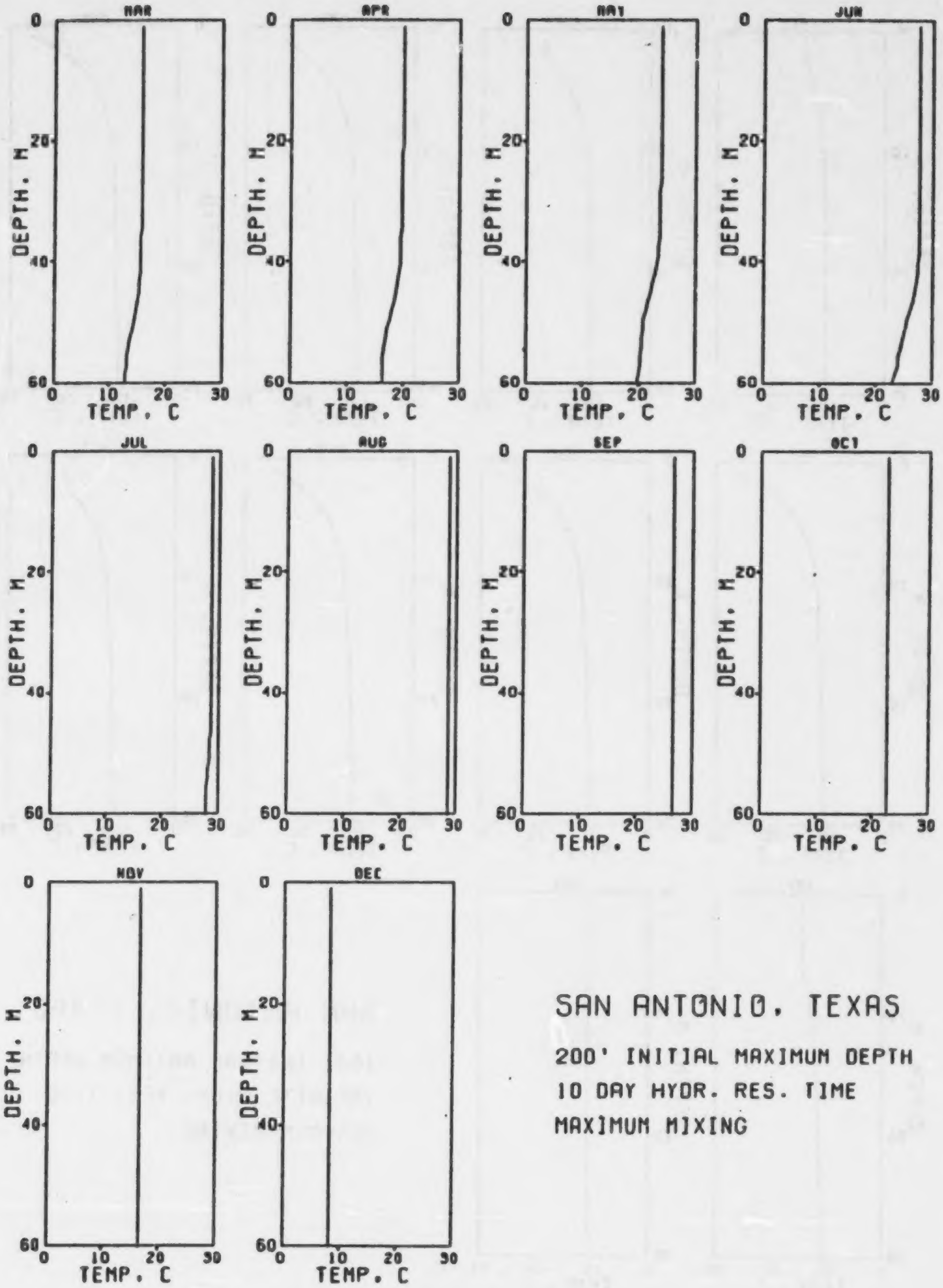
SAN ANTONIO, TEXAS

100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



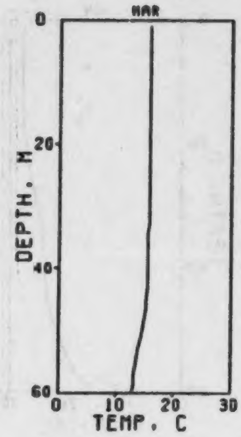
SAN ANTONIO, TEXAS

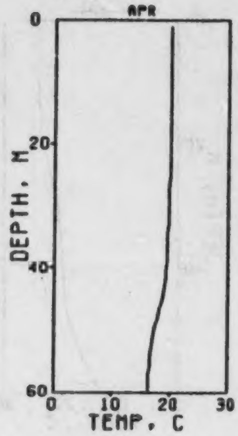
100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING

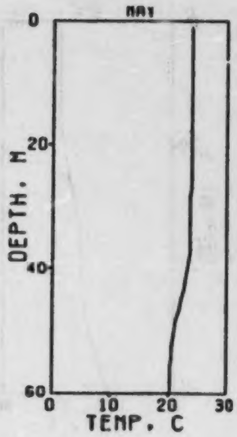


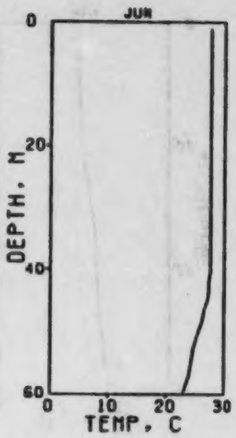
SAN ANTONIO, TEXAS

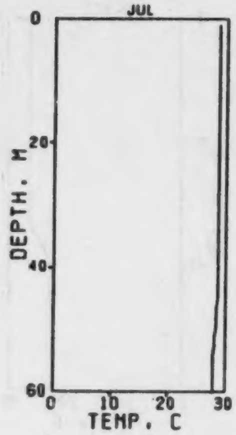
200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING

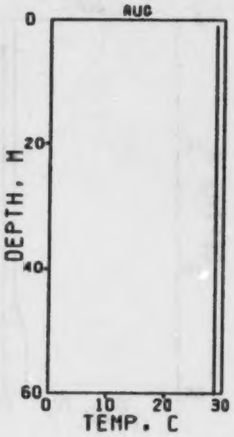


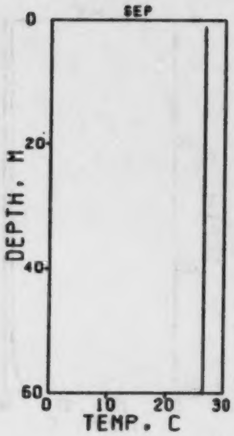


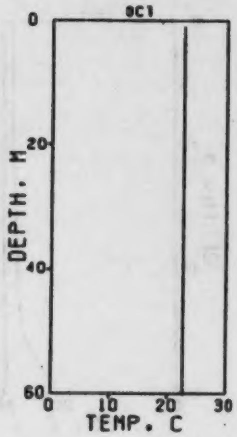


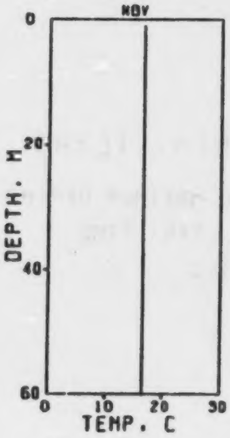


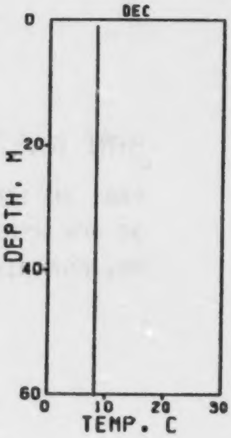






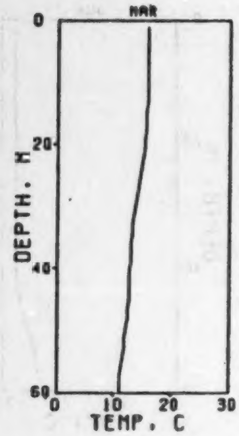


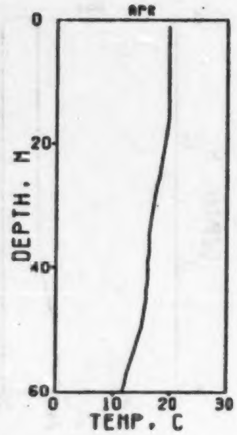


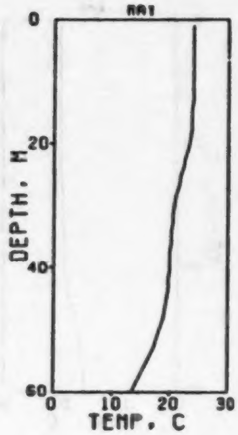


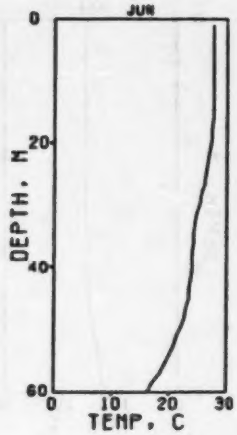
SAN ANTONIO, TEXAS

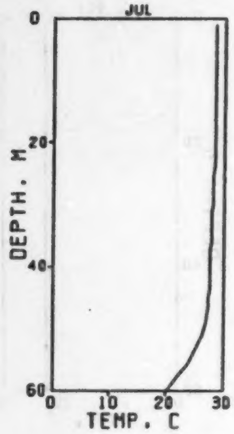
200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING

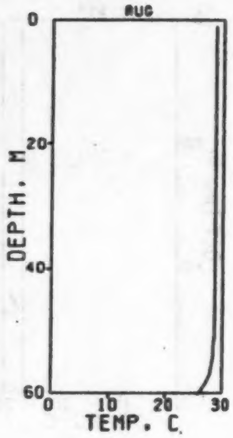


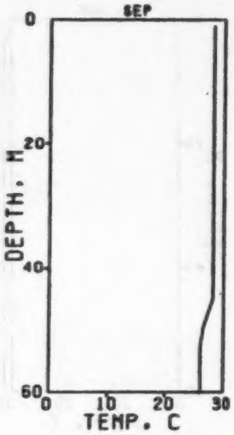


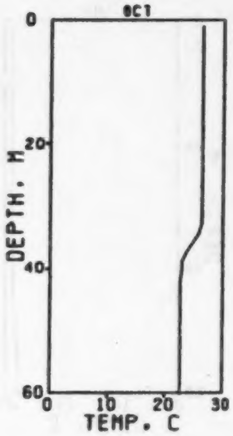


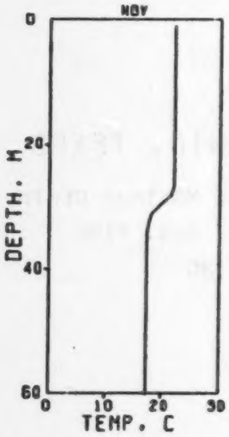


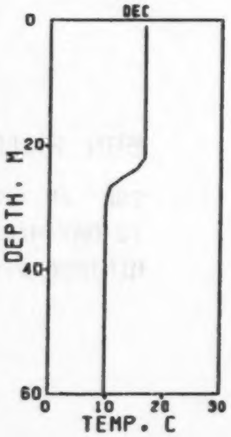






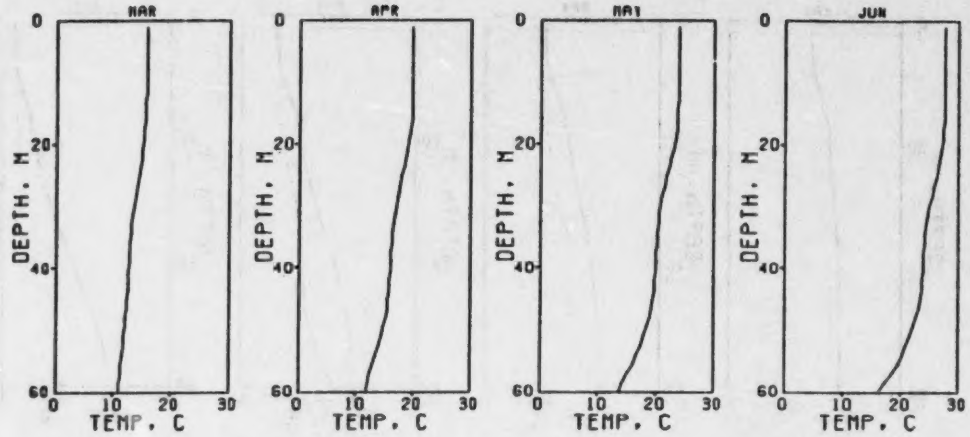


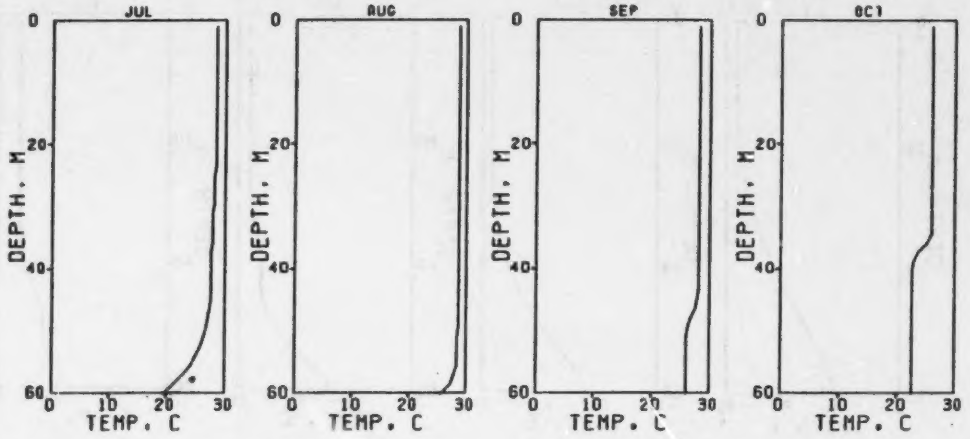


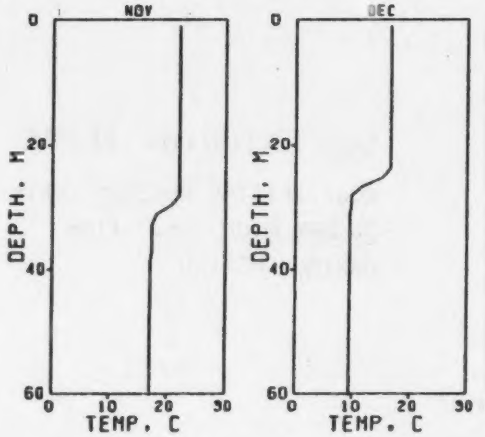


SAN ANTONIO, TEXAS

200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING

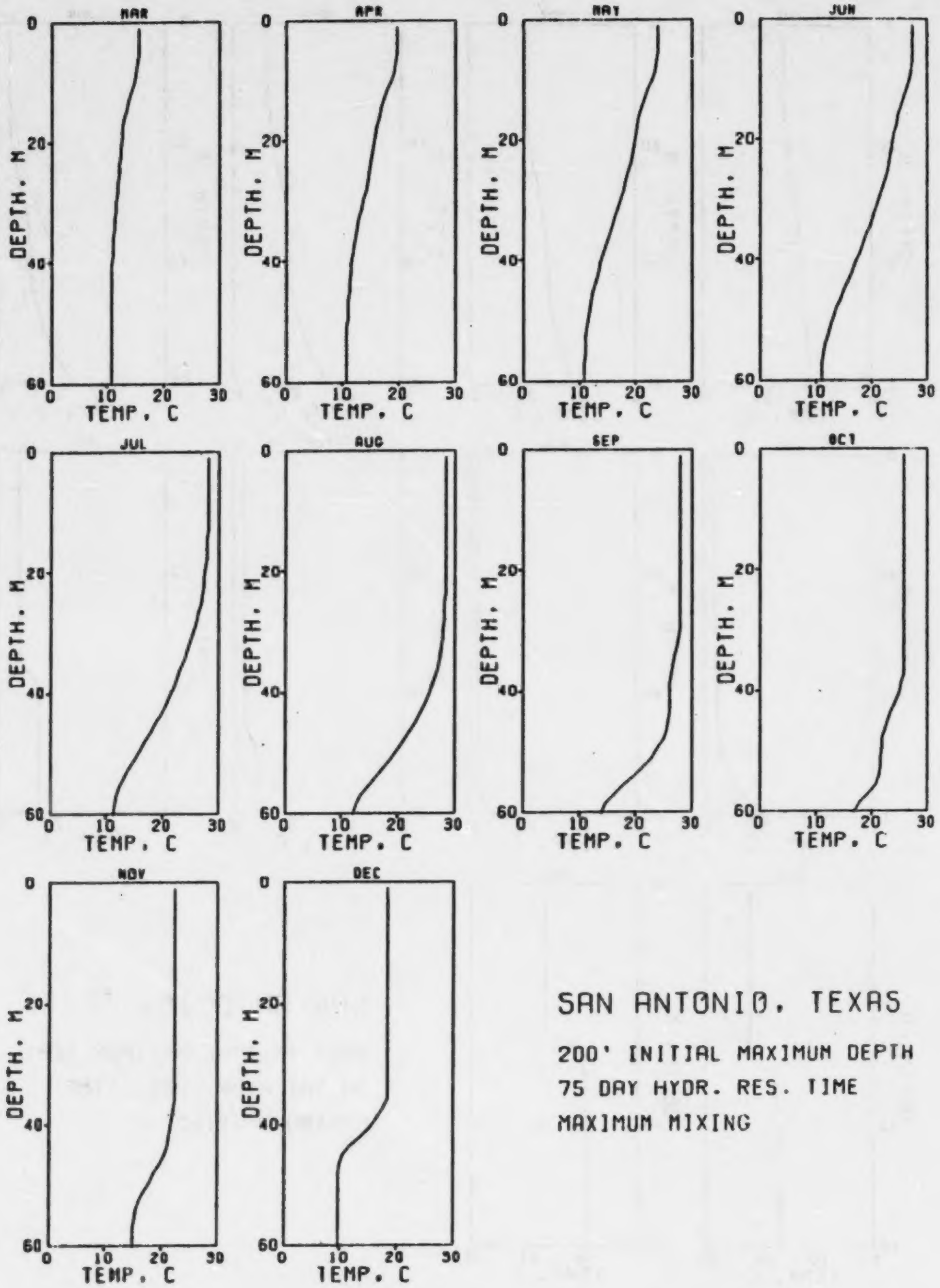






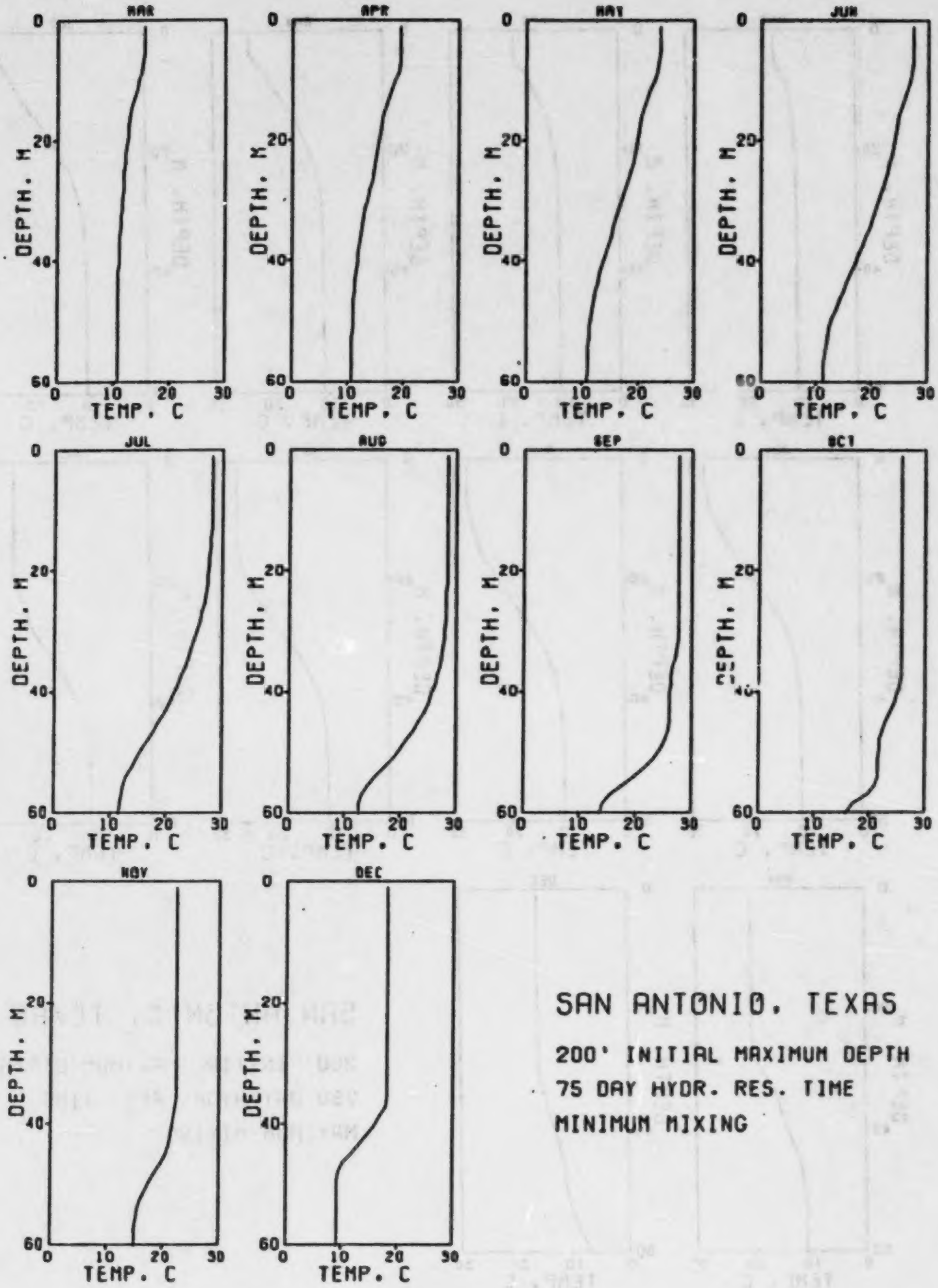
SAN ANTONIO, TEXAS

200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



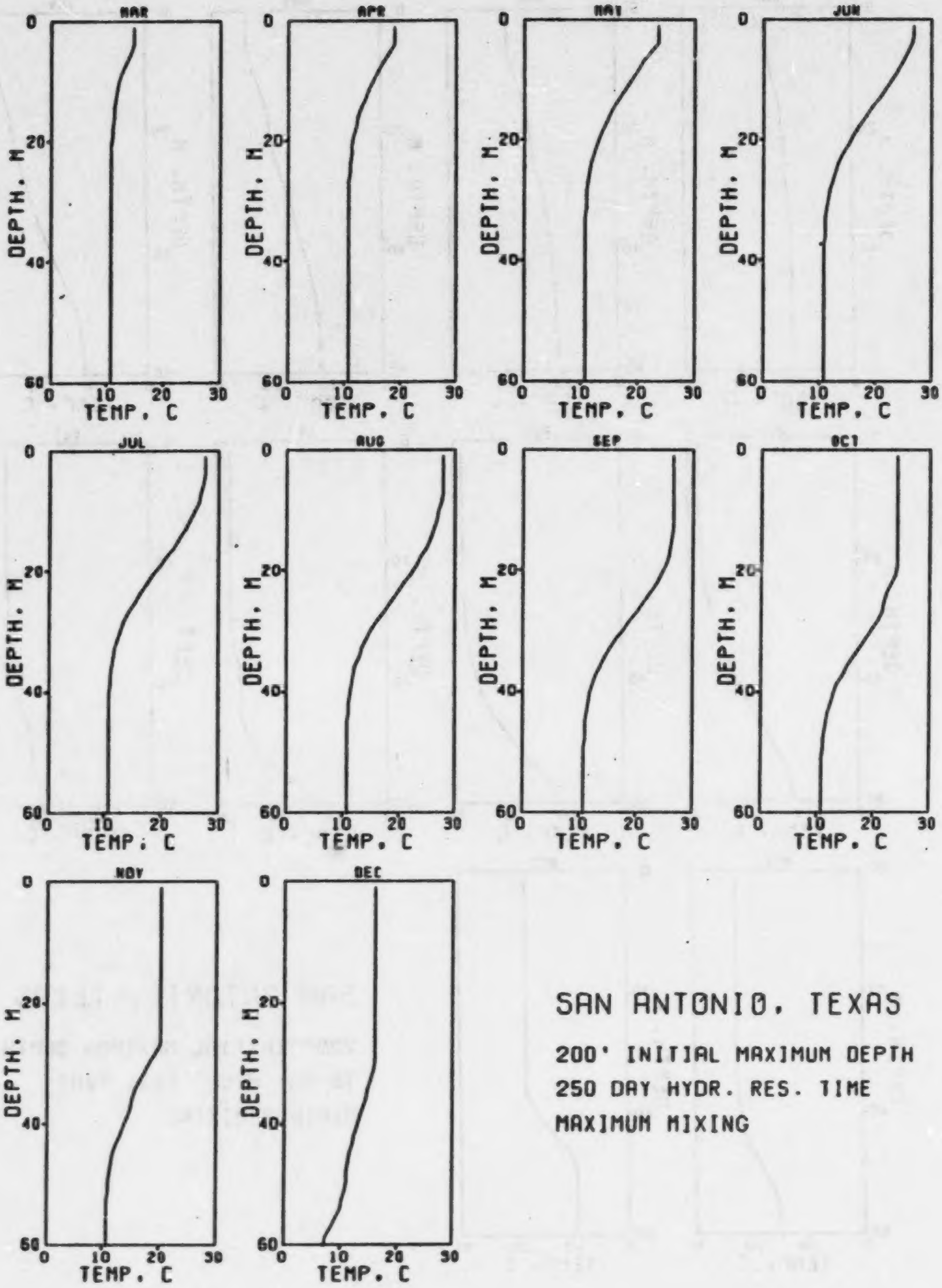
SAN ANTONIO, TEXAS

200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



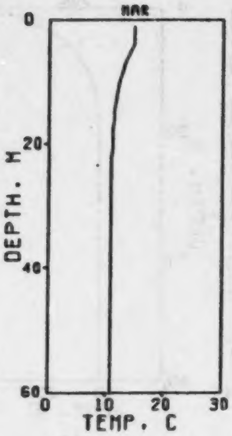
SAN ANTONIO, TEXAS

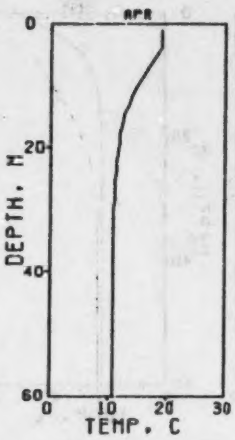
200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING

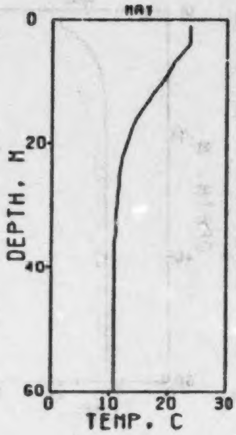


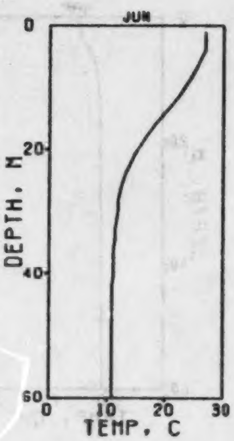
SAN ANTONIO, TEXAS

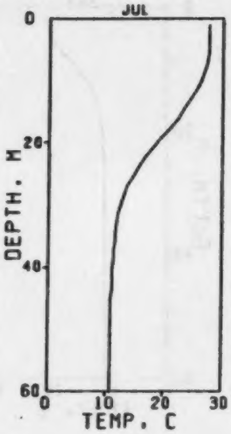
200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING

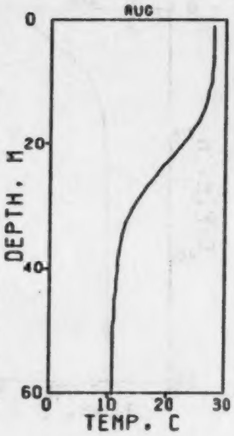


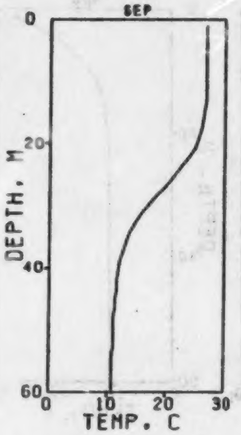


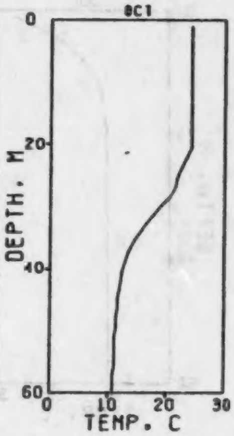


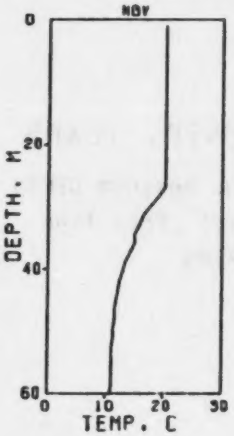


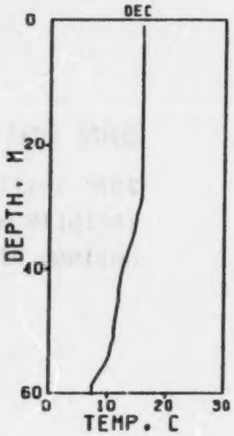






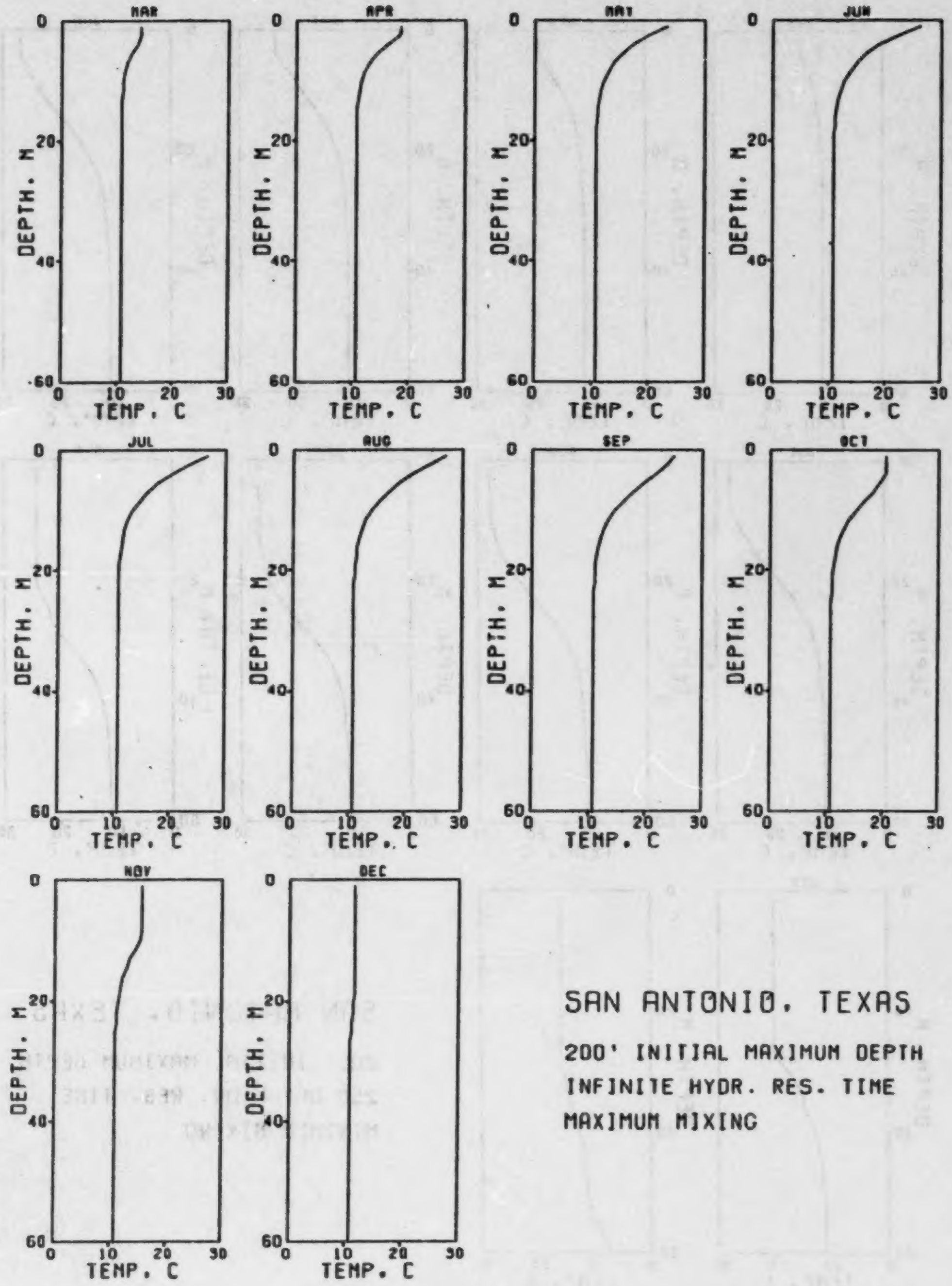






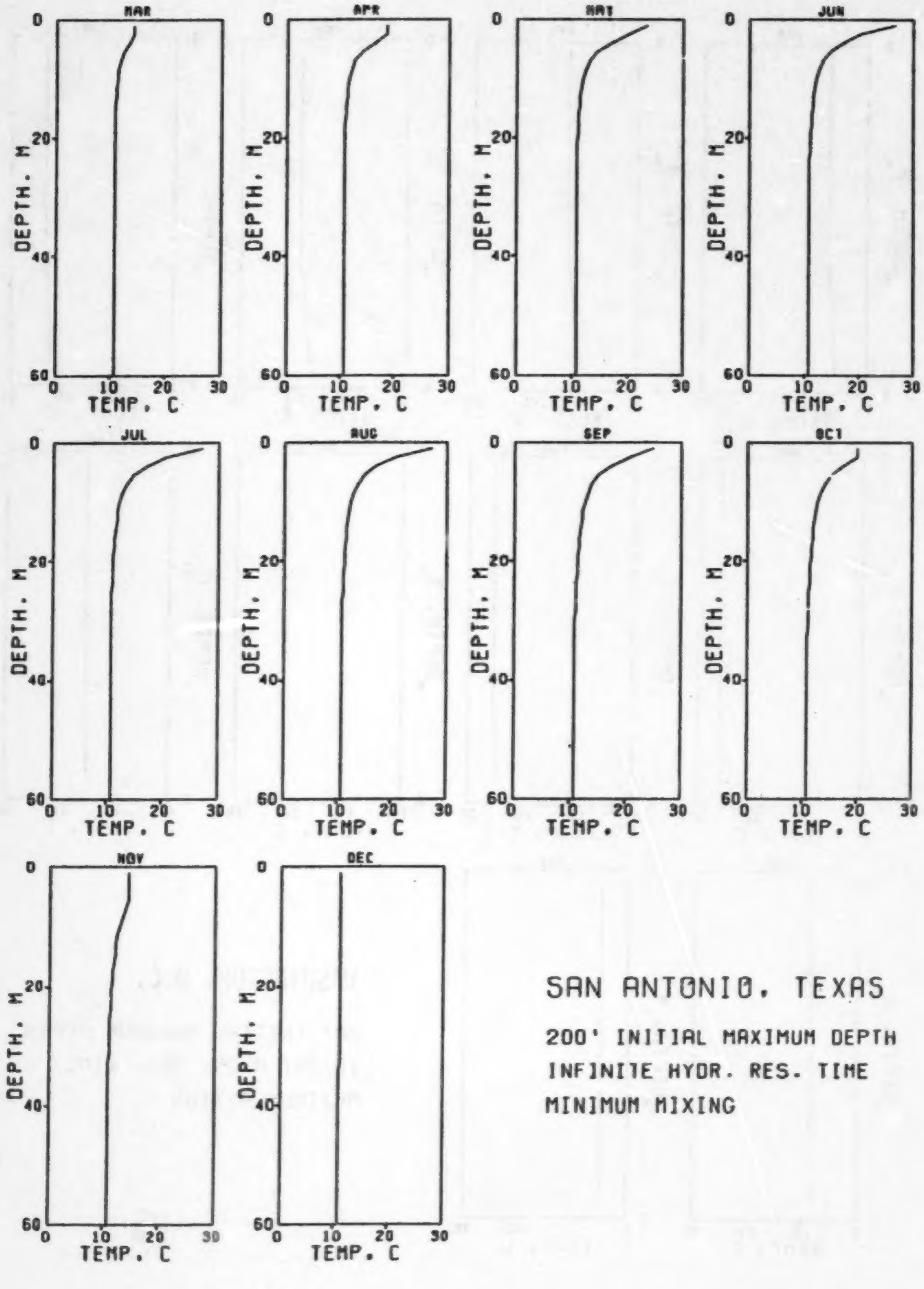
SAN ANTONIO, TEXAS

200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



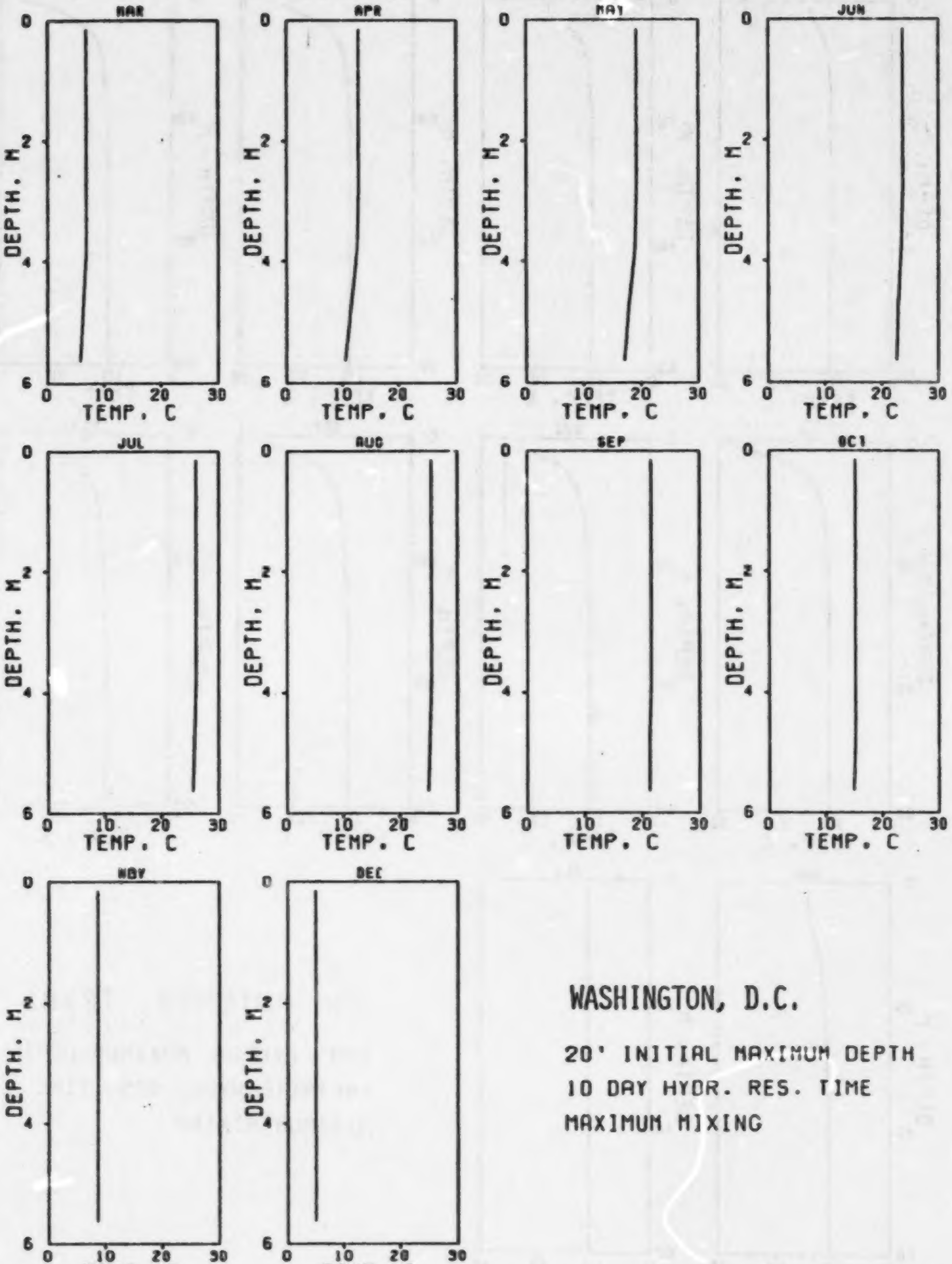
SAN ANTONIO, TEXAS

200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



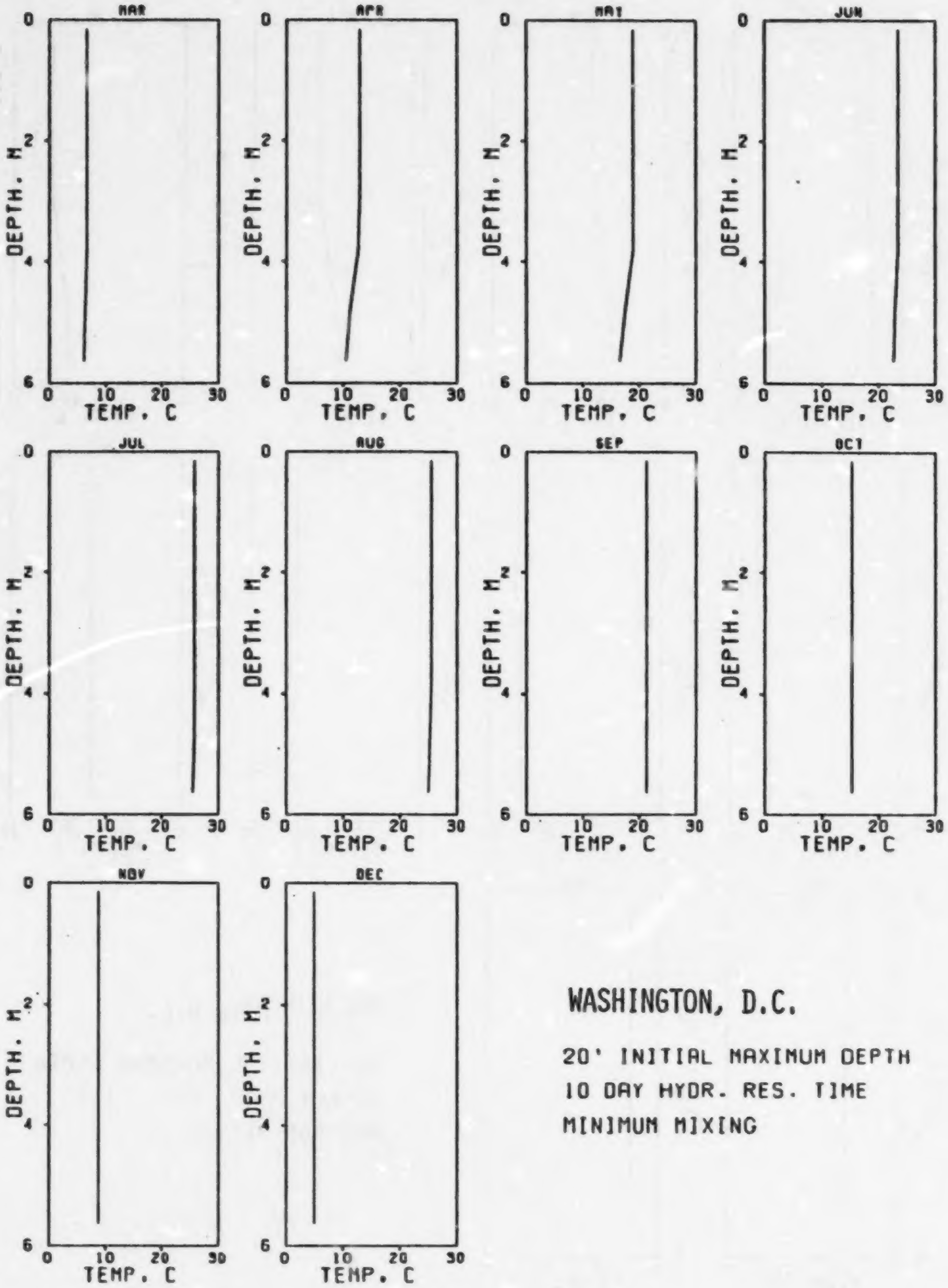
SAN ANTONIO, TEXAS

200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



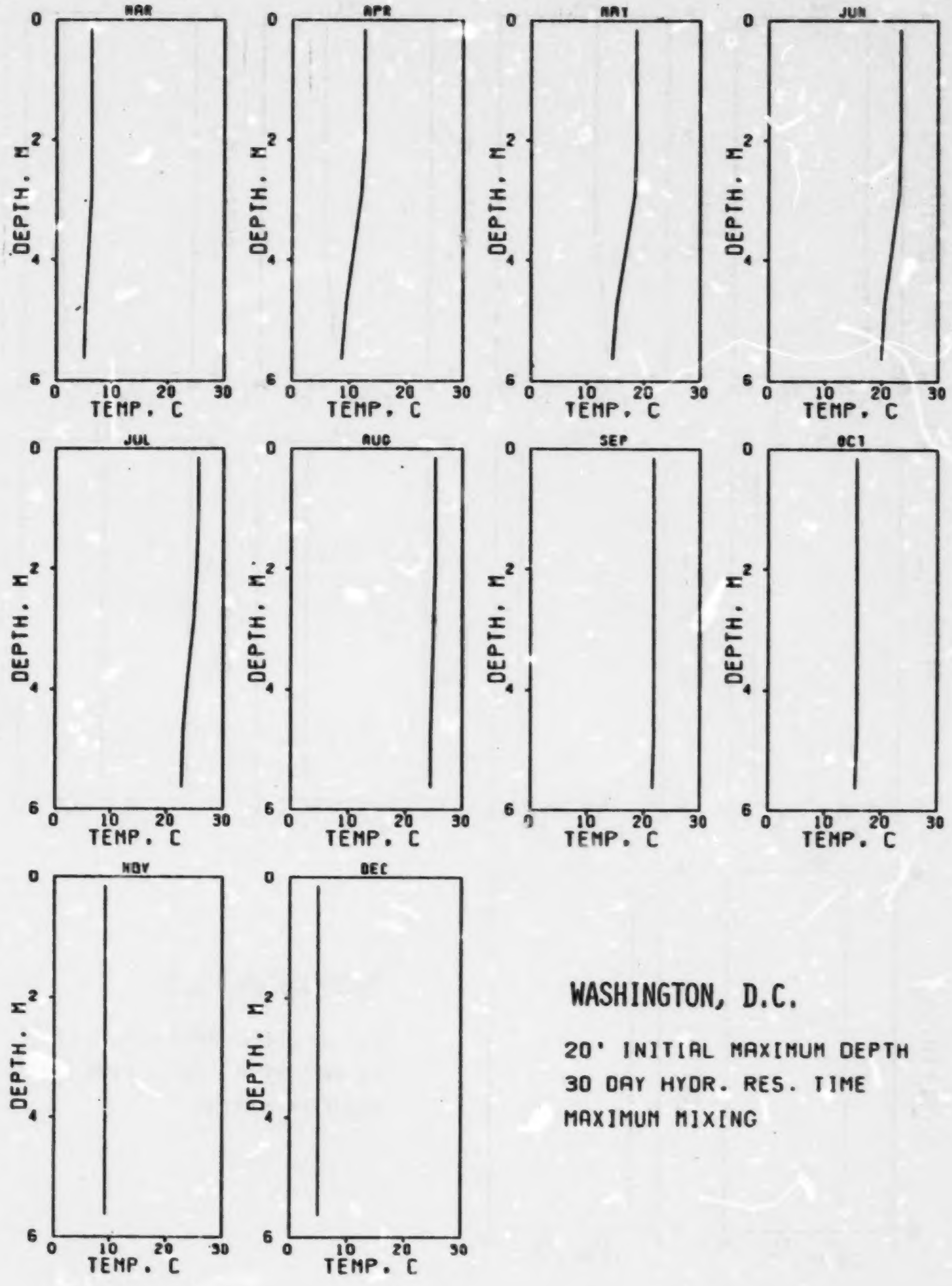
WASHINGTON, D.C.

20' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



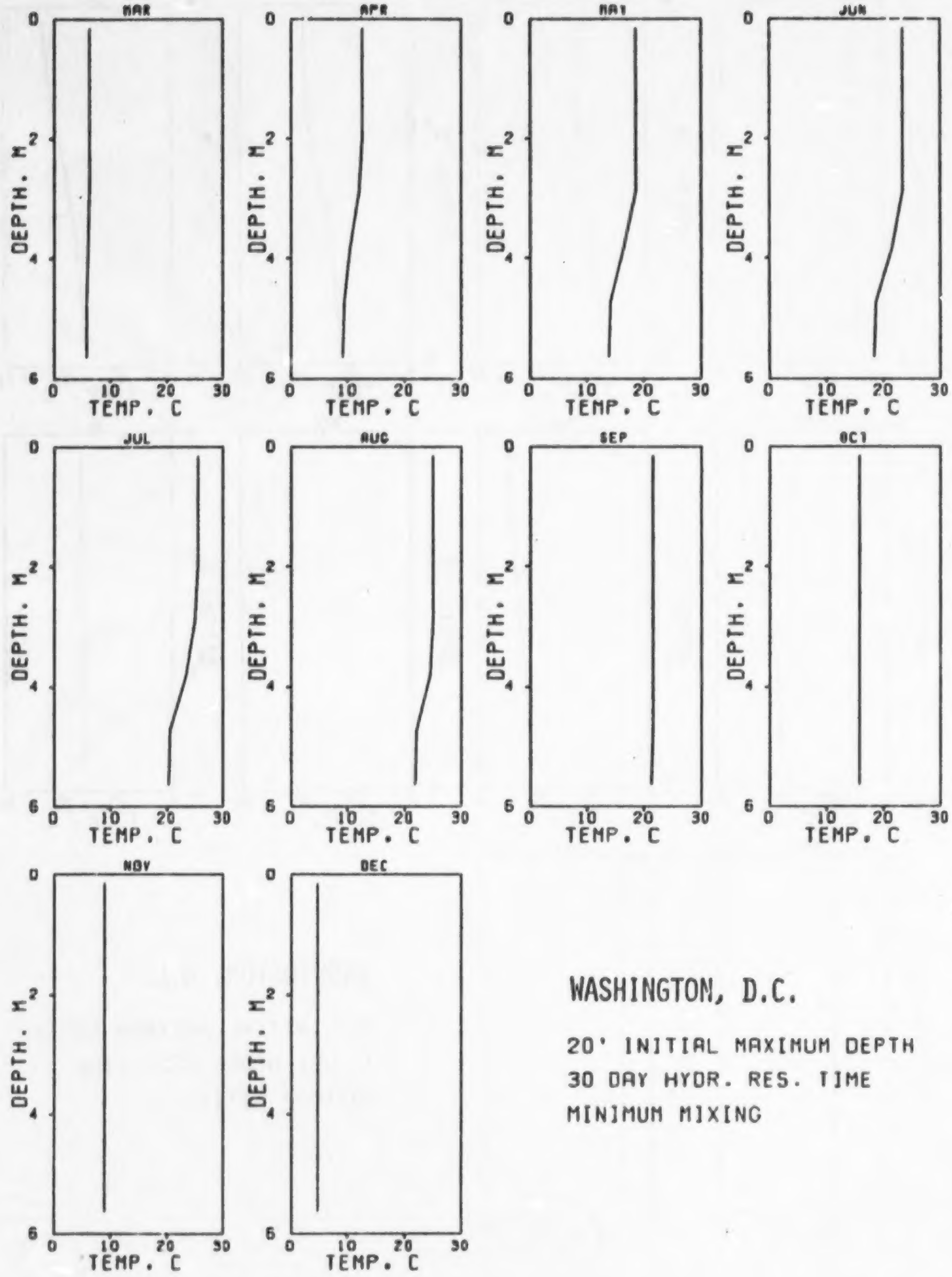
WASHINGTON, D.C.

20' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



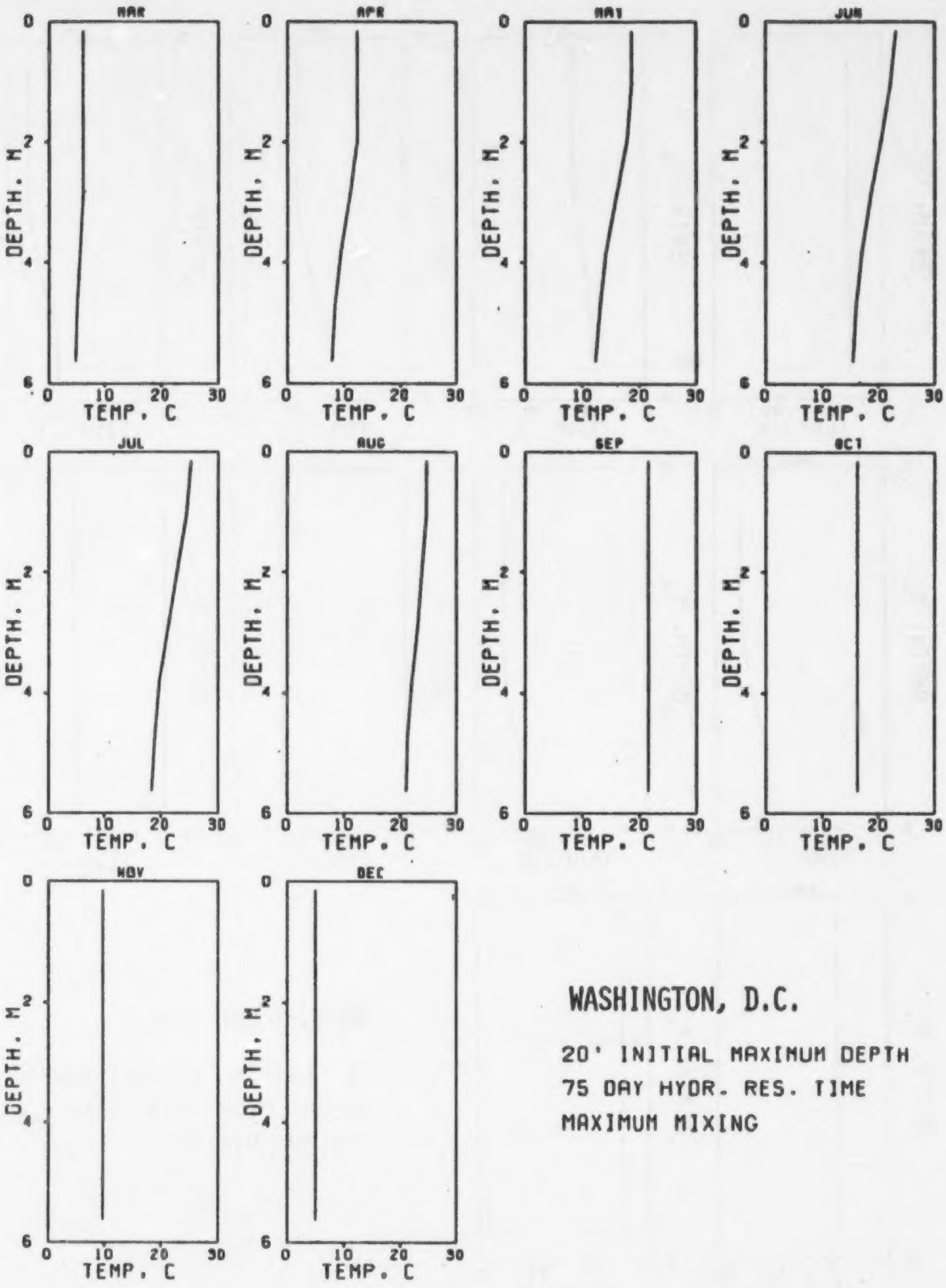
WASHINGTON, D.C.

20' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



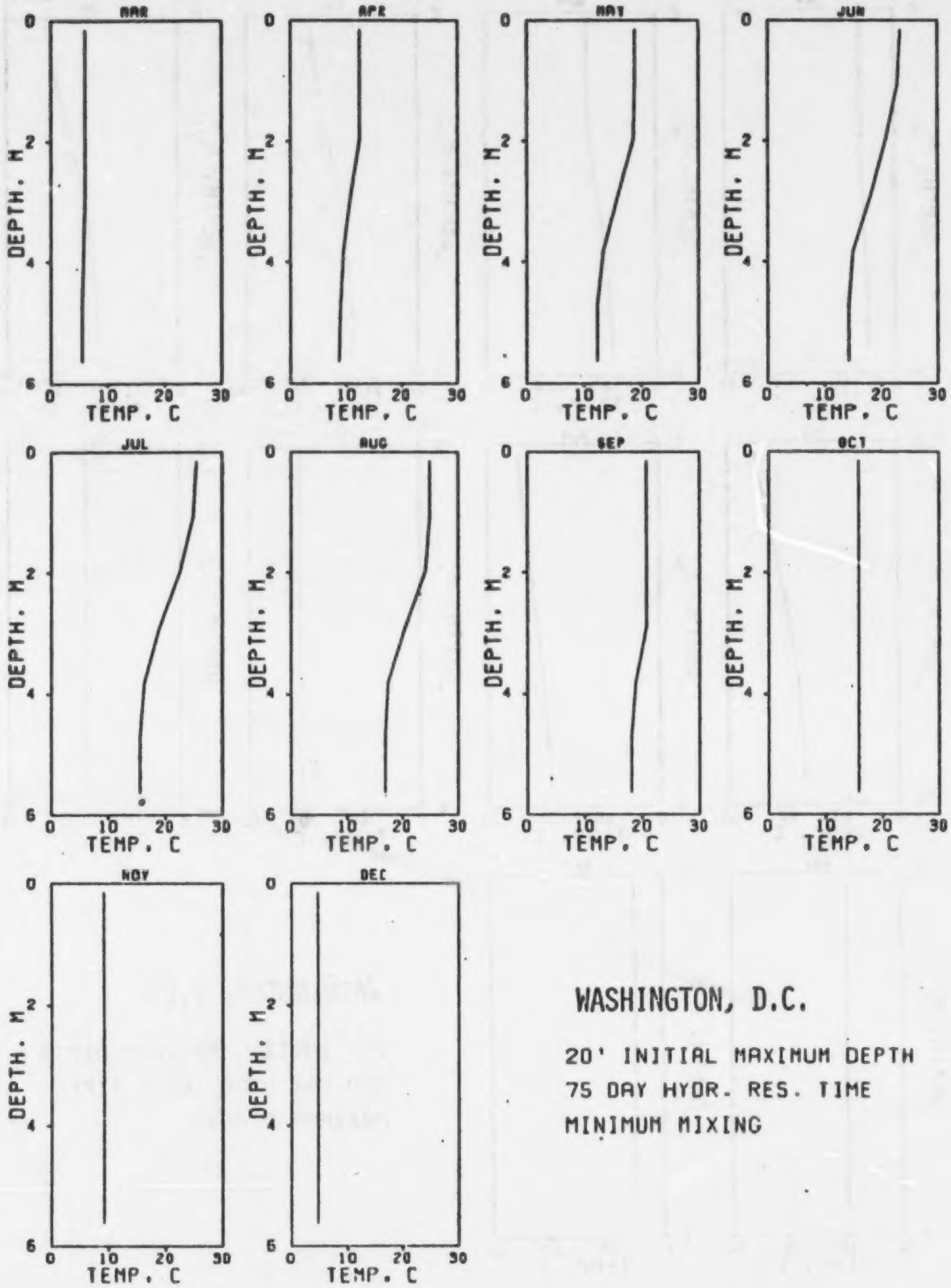
WASHINGTON, D.C.

20' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



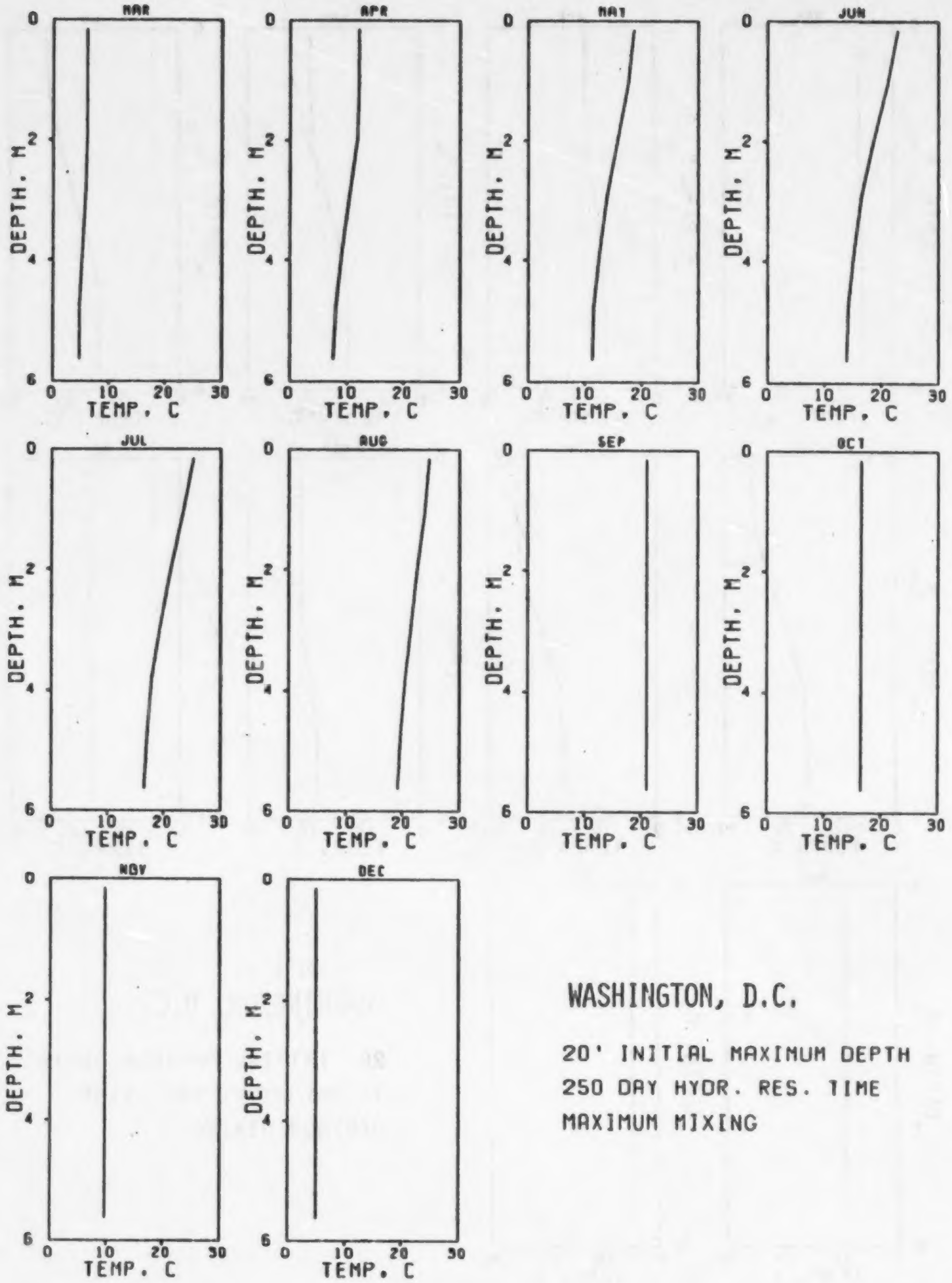
WASHINGTON, D.C.

20' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



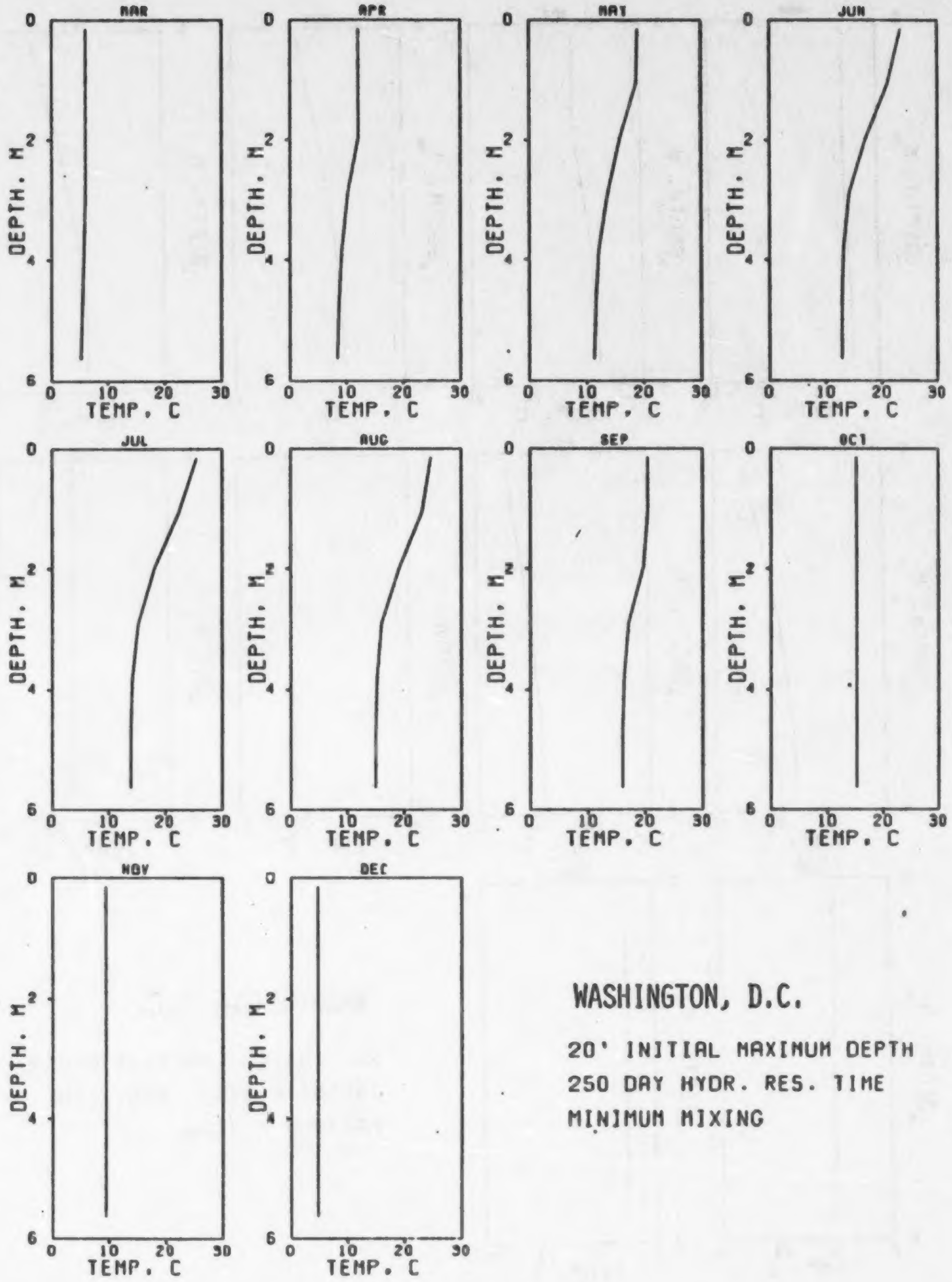
WASHINGTON, D.C.

20' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



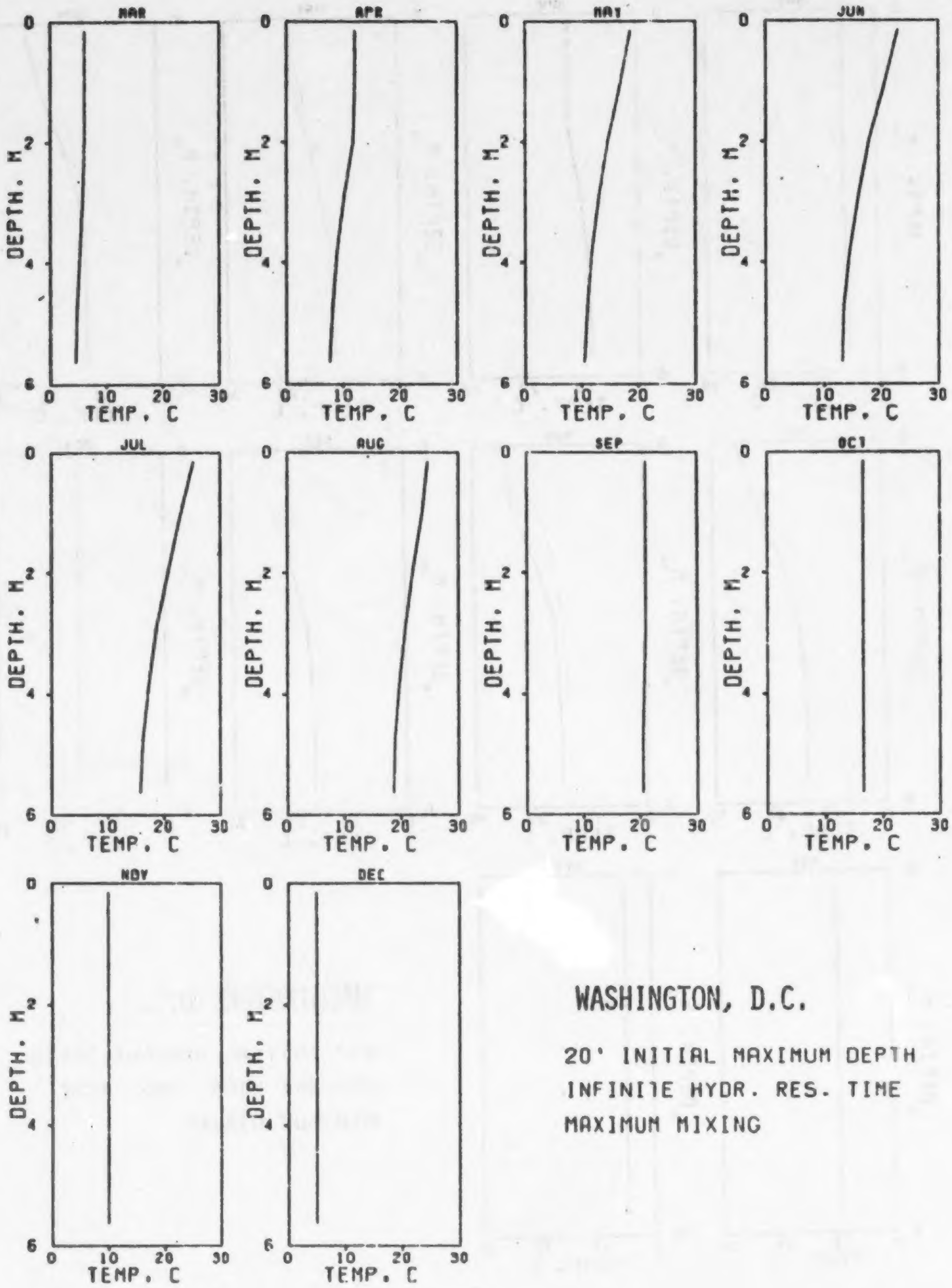
WASHINGTON, D.C.

20' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



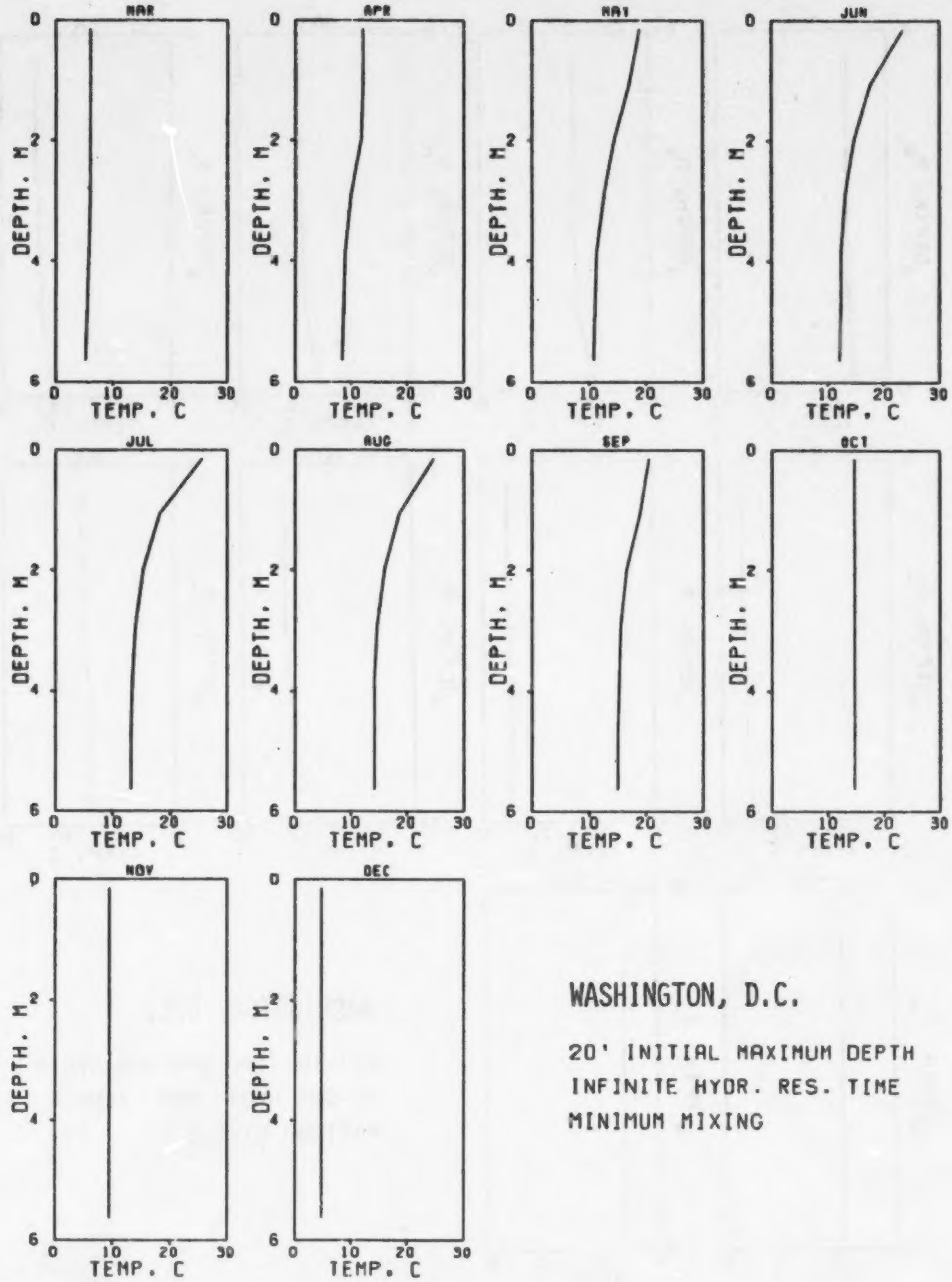
WASHINGTON, D.C.

20' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



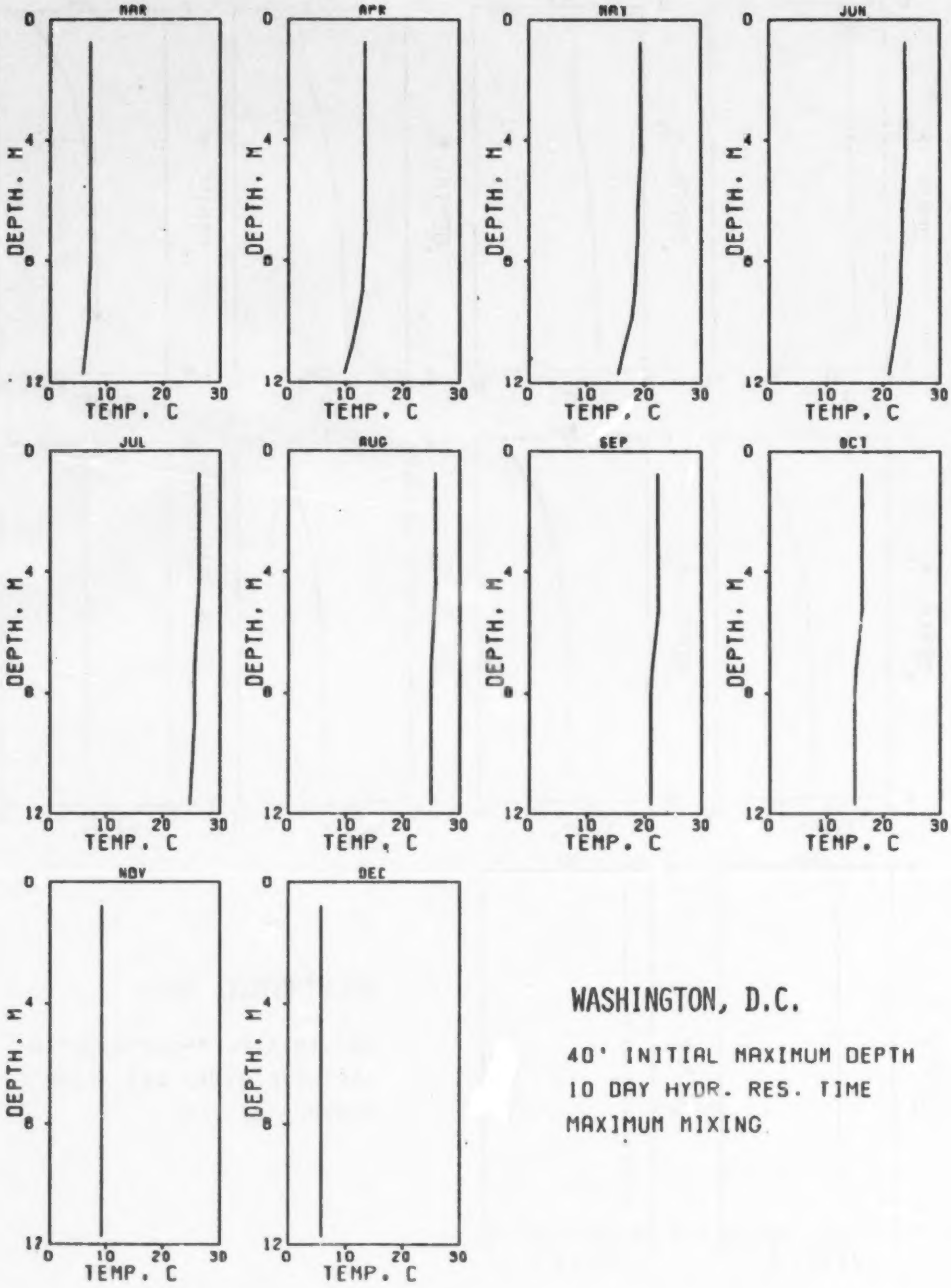
WASHINGTON, D.C.

20' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



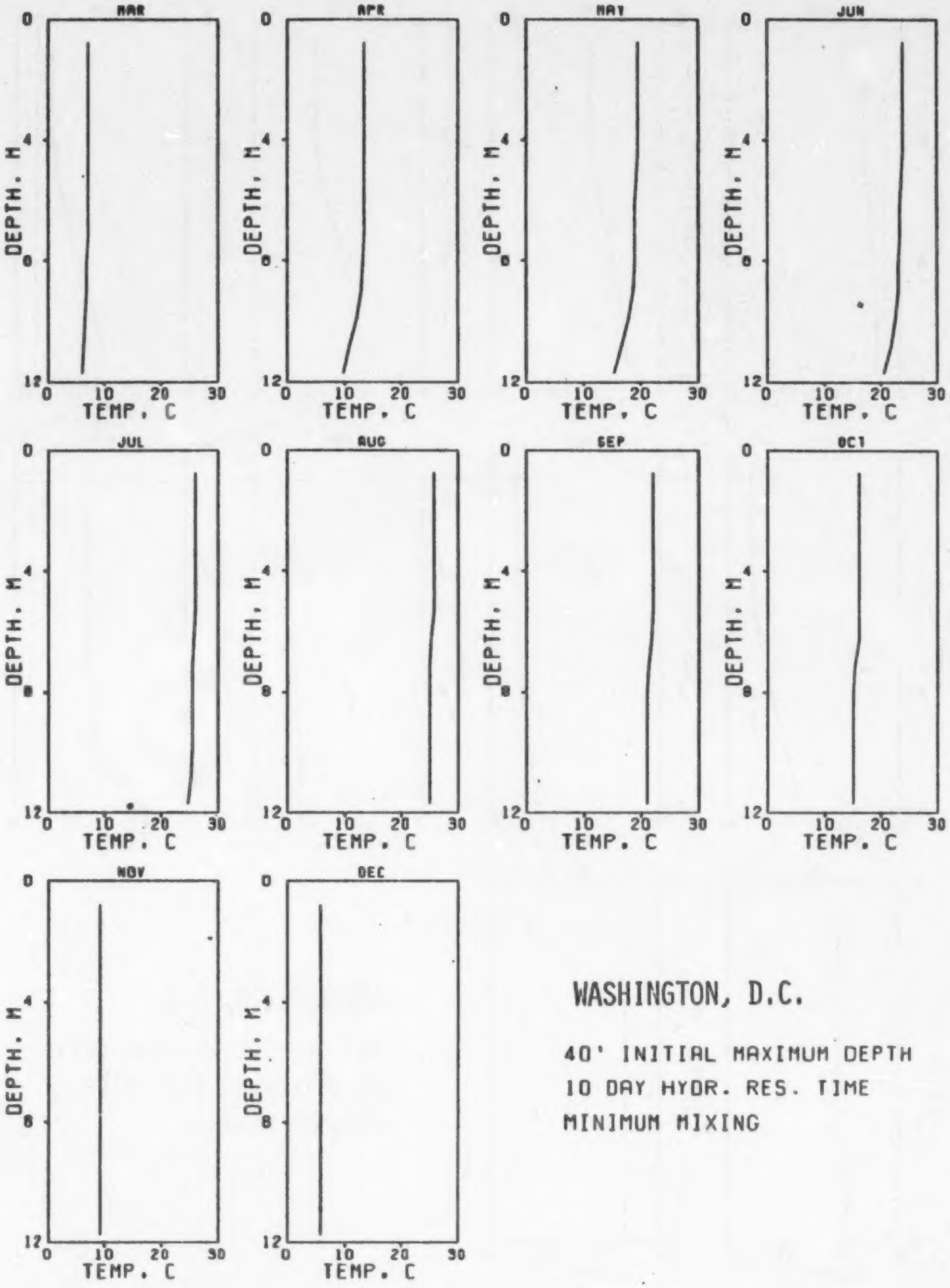
WASHINGTON, D.C.

20' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



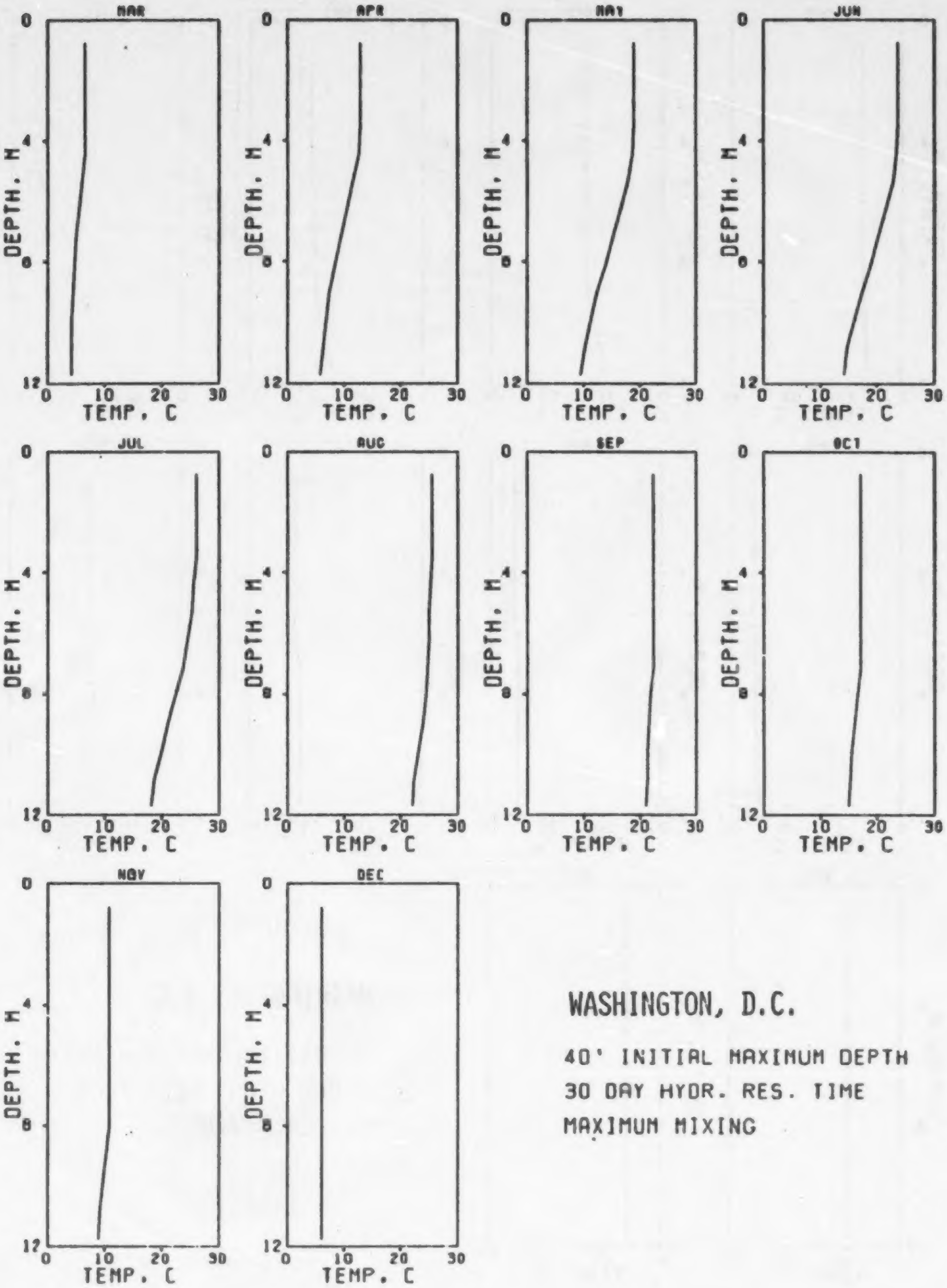
WASHINGTON, D.C.

40' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



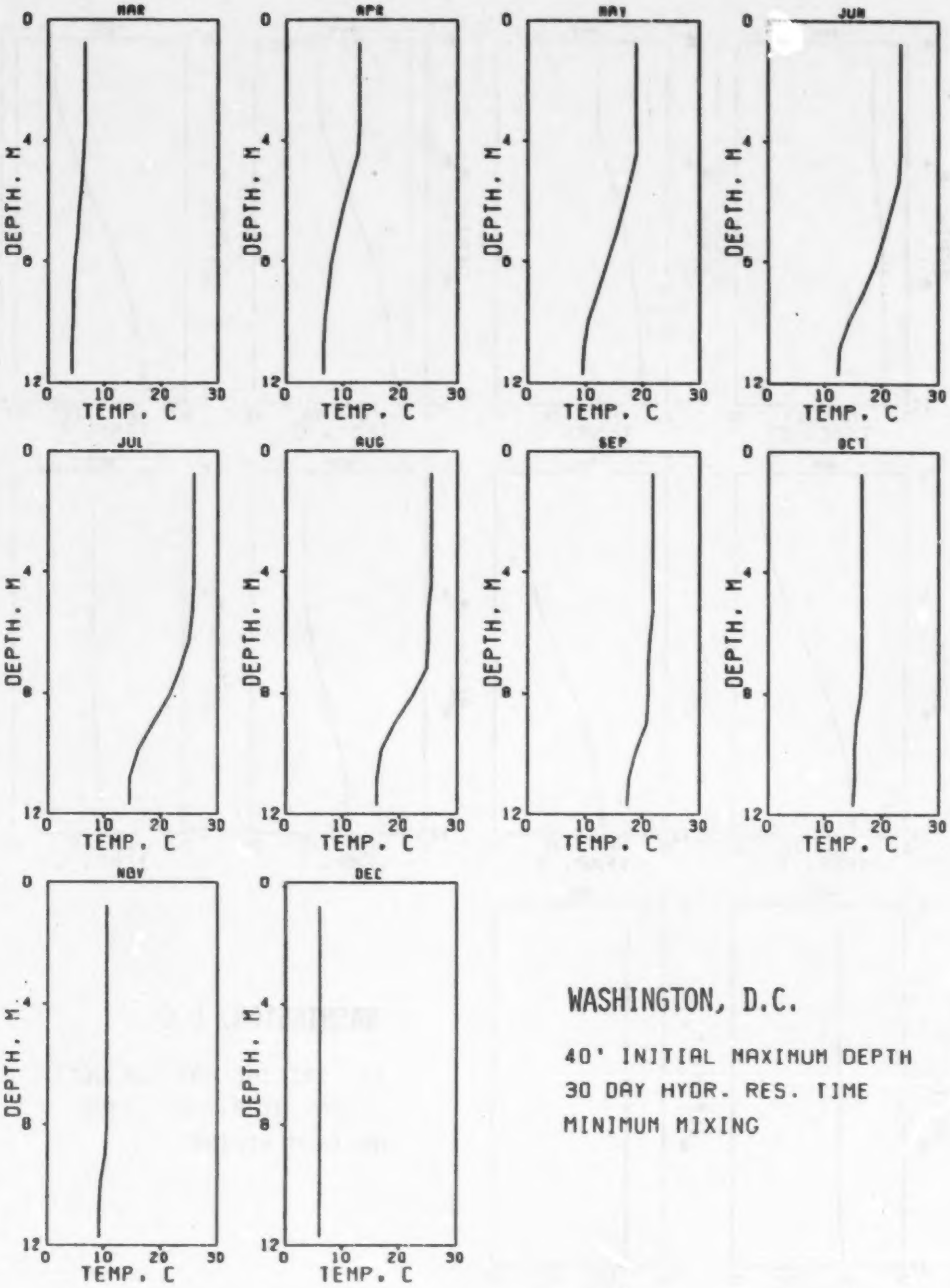
WASHINGTON, D.C.

40' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



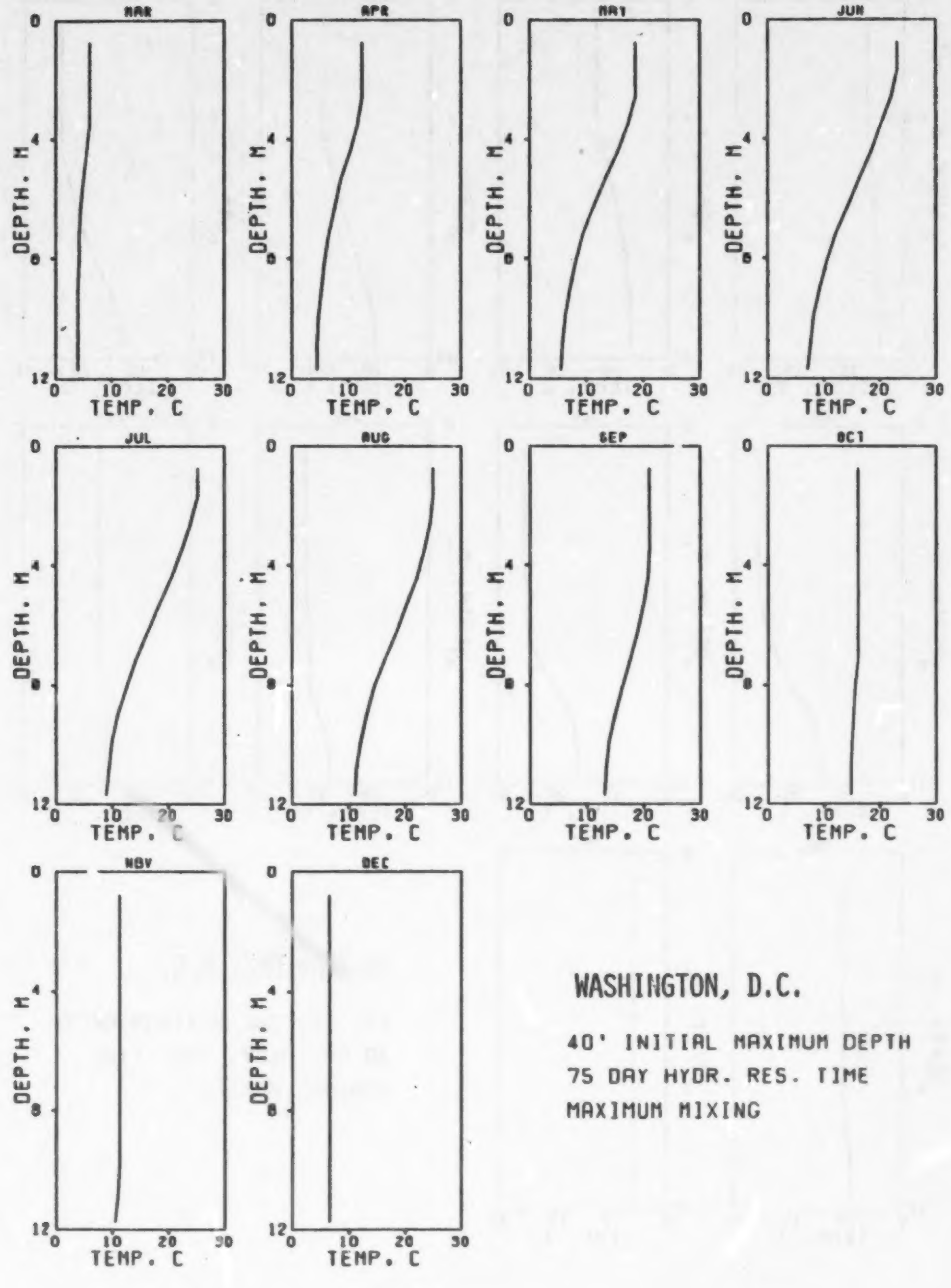
WASHINGTON, D.C.

40' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



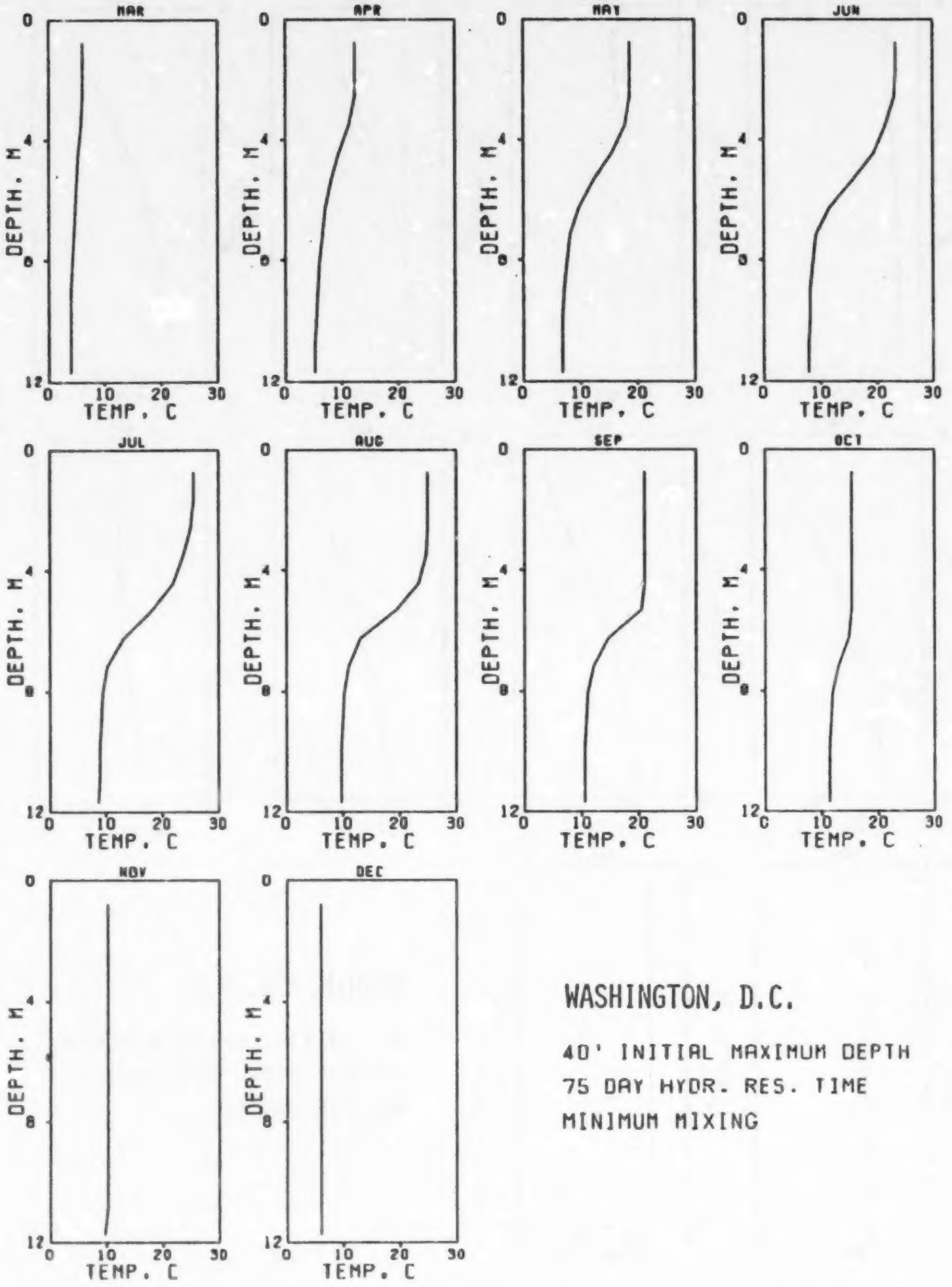
WASHINGTON, D.C.

40' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



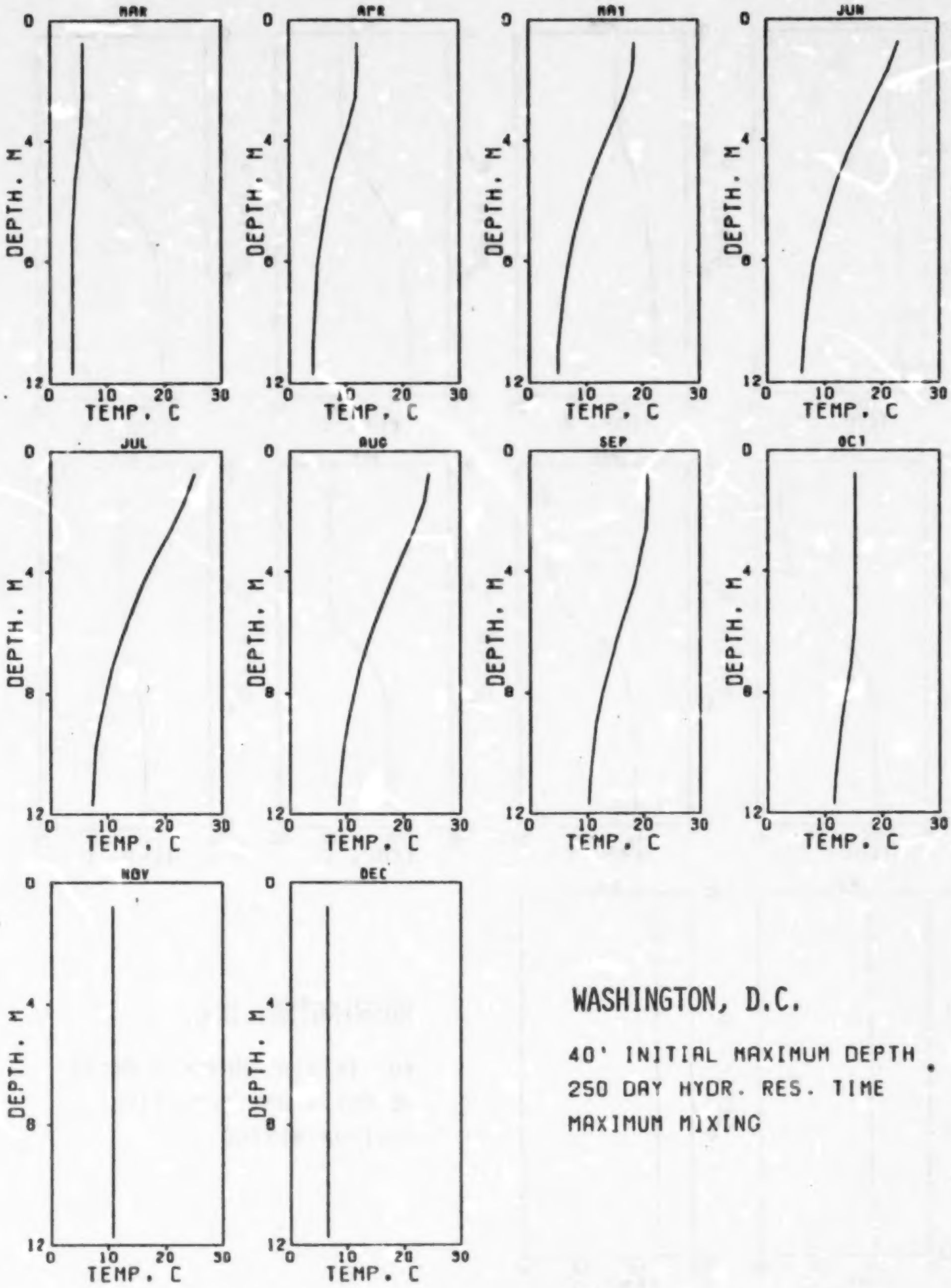
WASHINGTON, D.C.

40' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



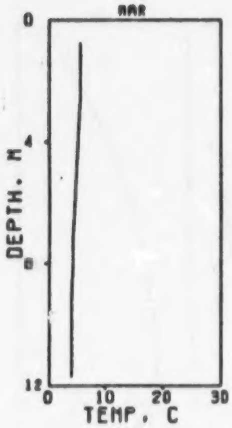
WASHINGTON, D.C.

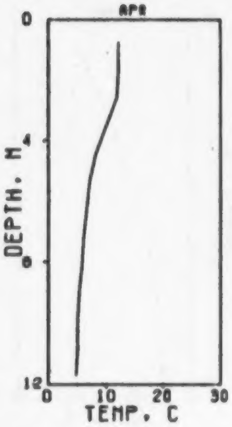
40' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING

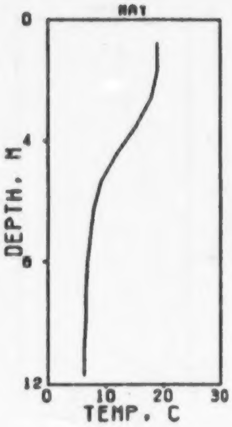


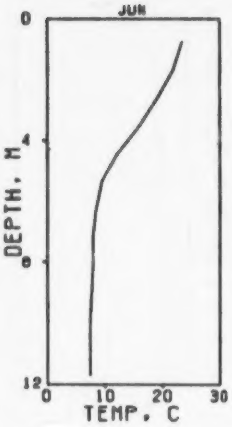
WASHINGTON, D.C.

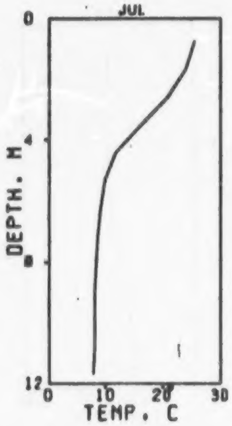
40' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING

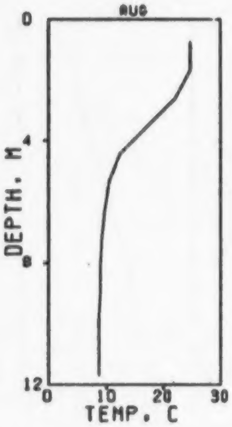


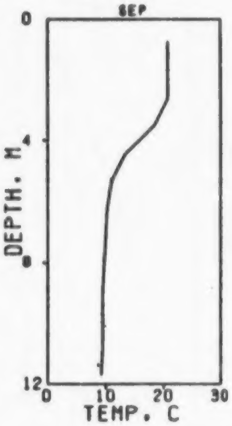


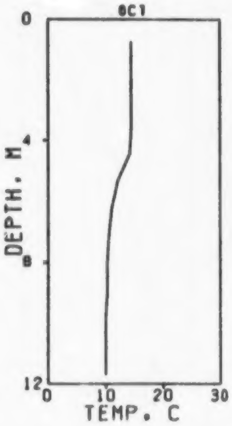


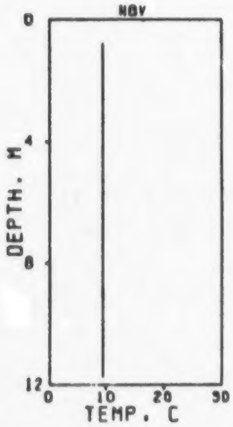


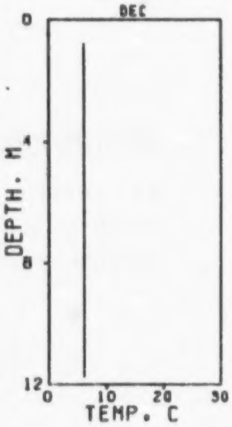






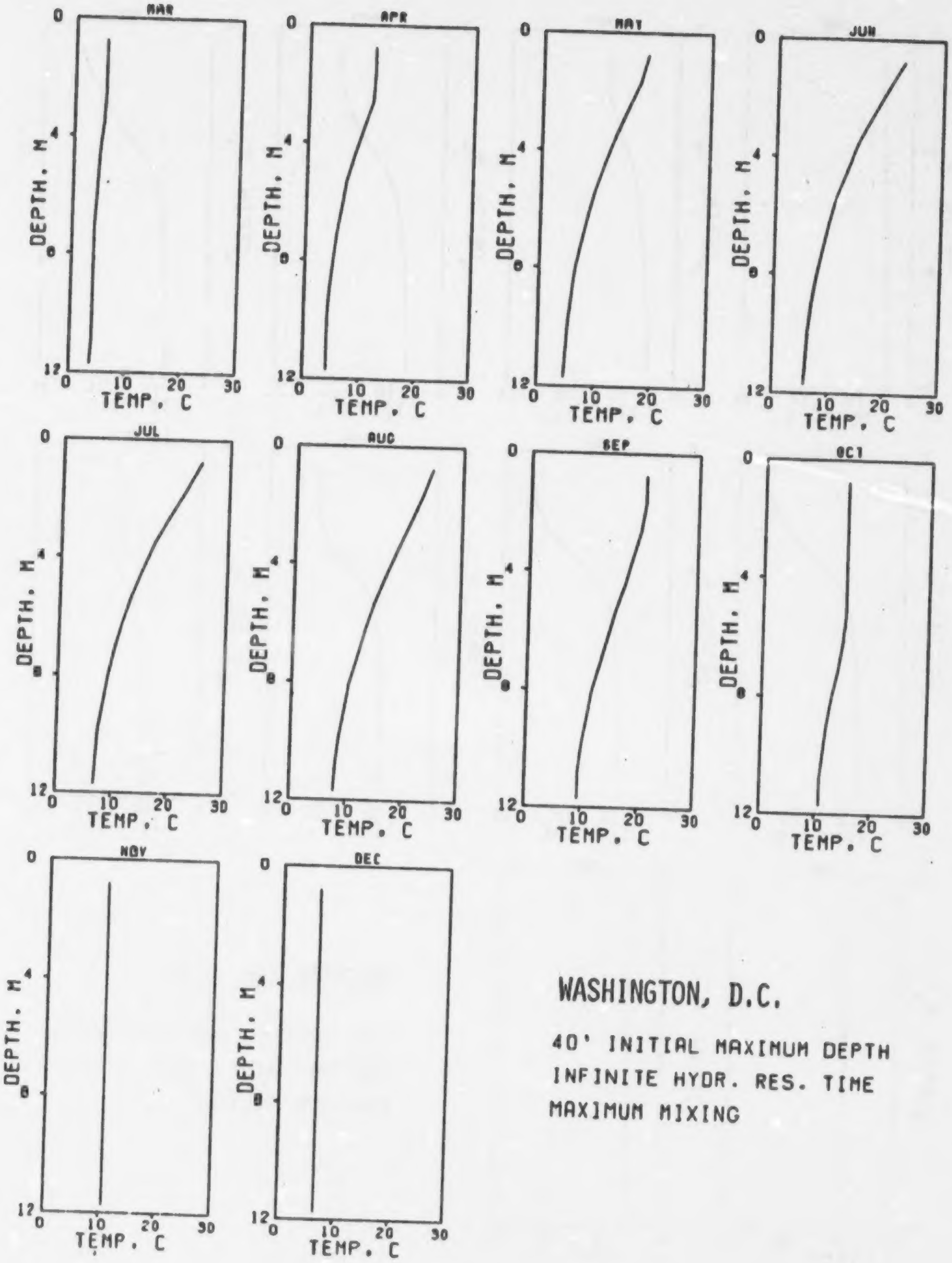






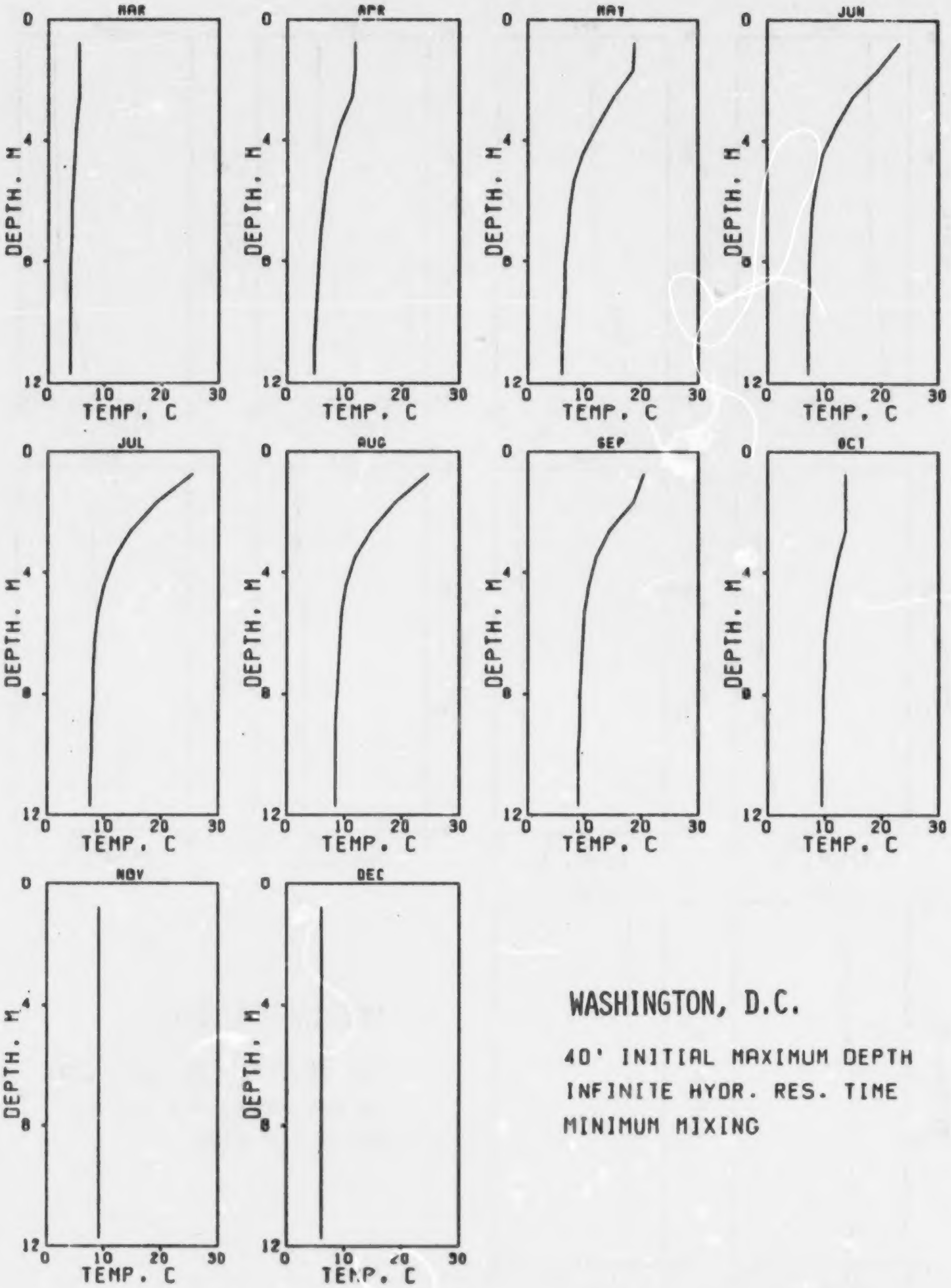
WASHINGTON, D.C.

40' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



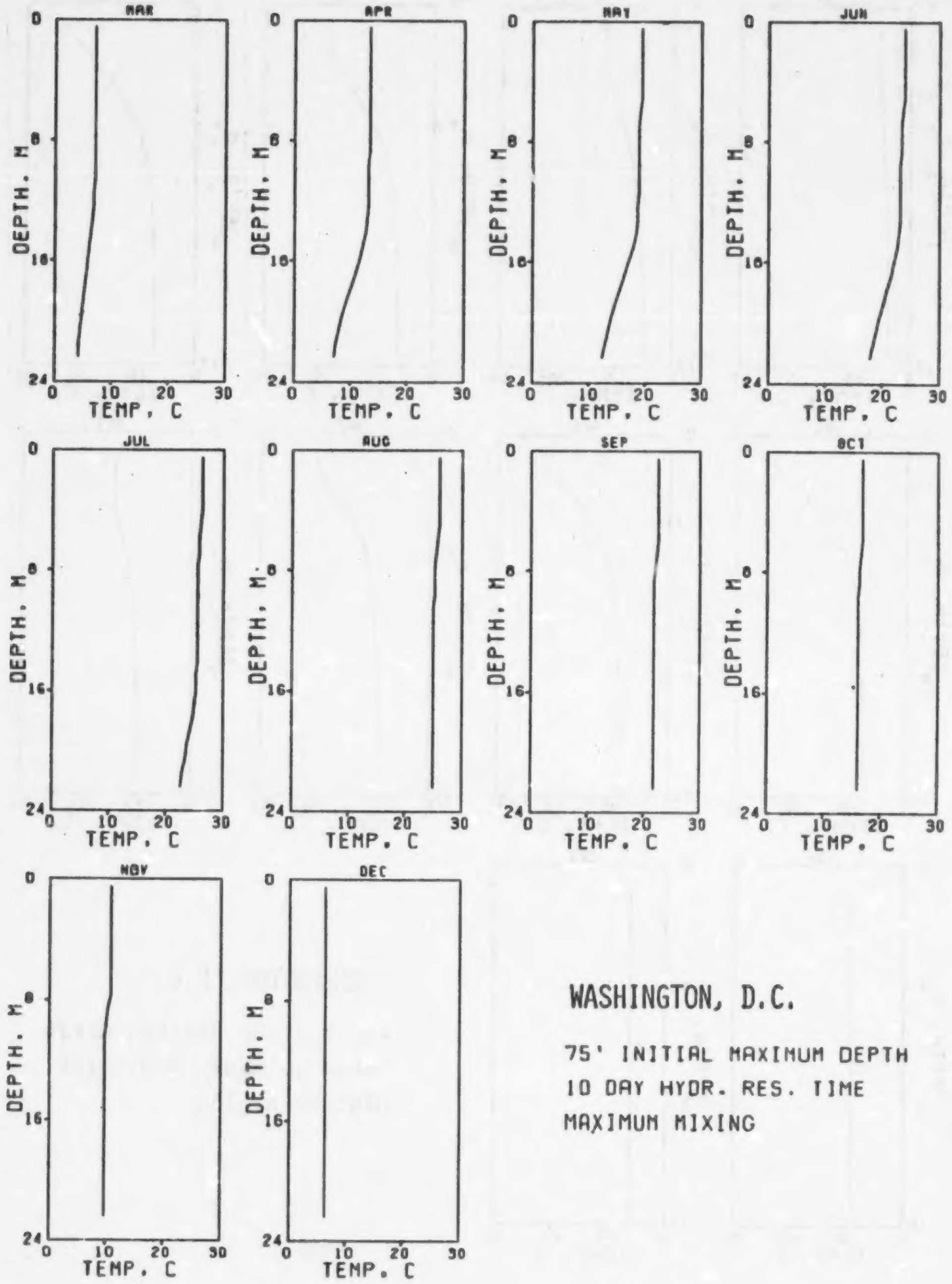
WASHINGTON, D.C.

40' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



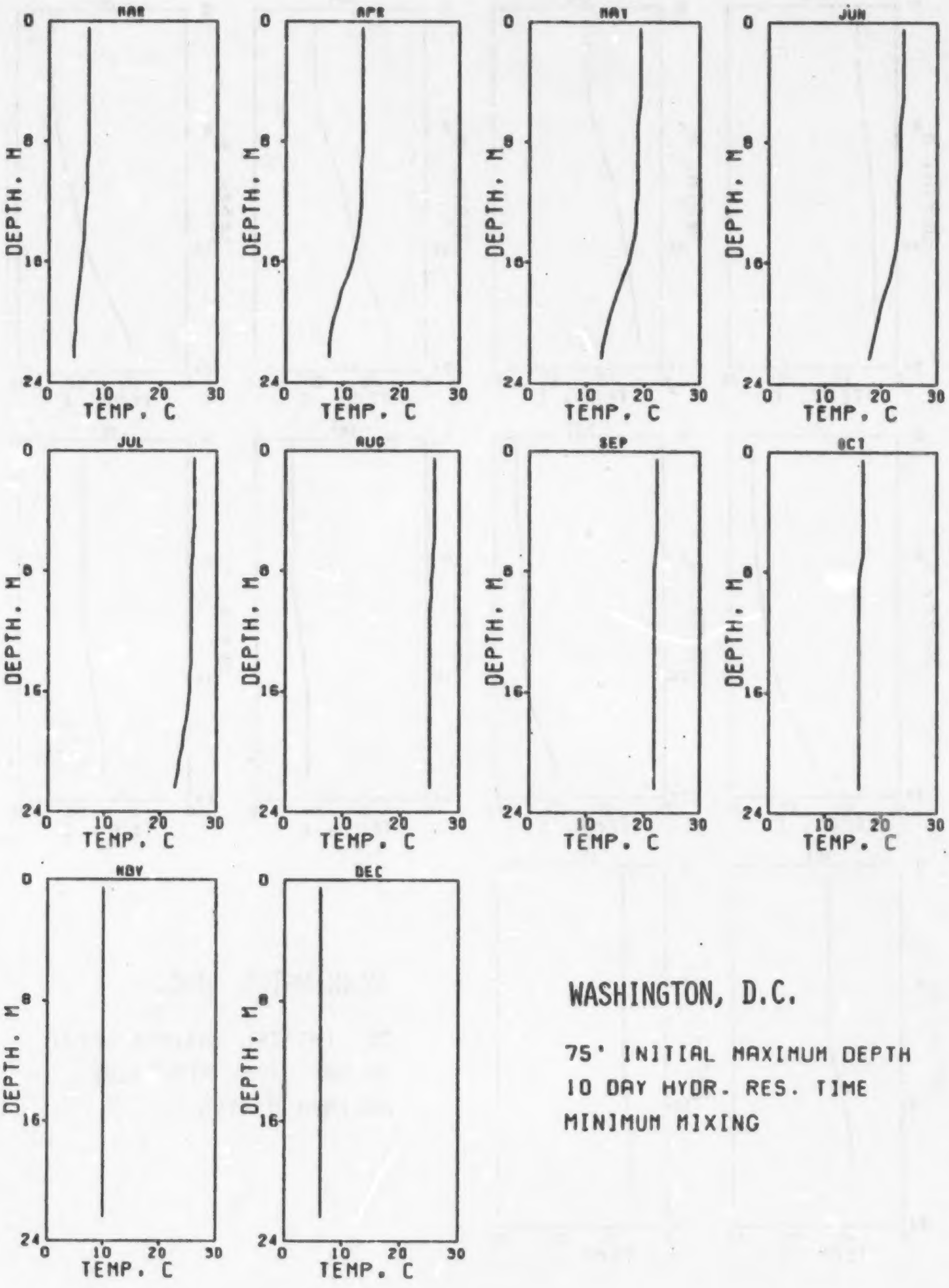
WASHINGTON, D.C.

40' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



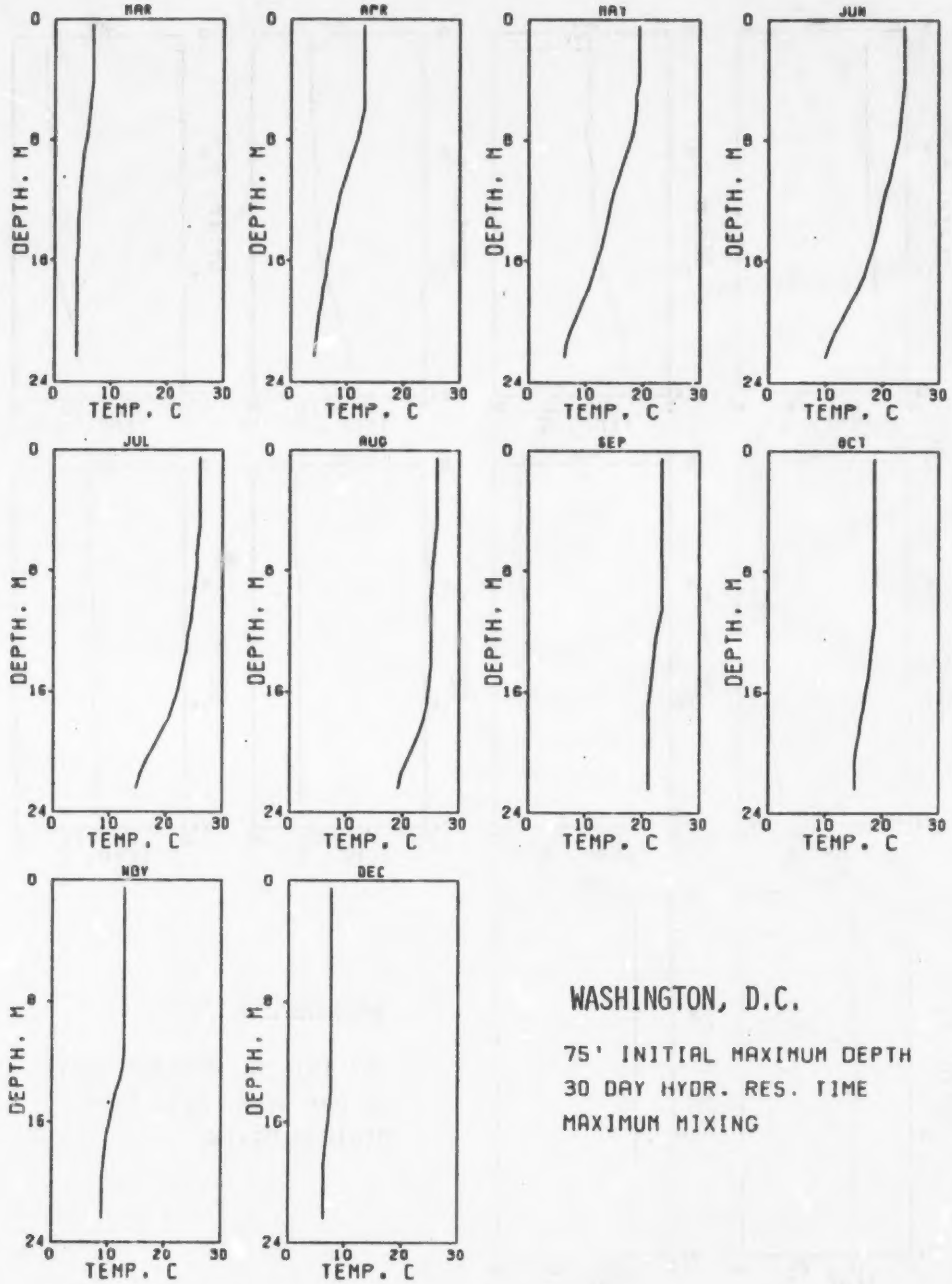
WASHINGTON, D.C.

75' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



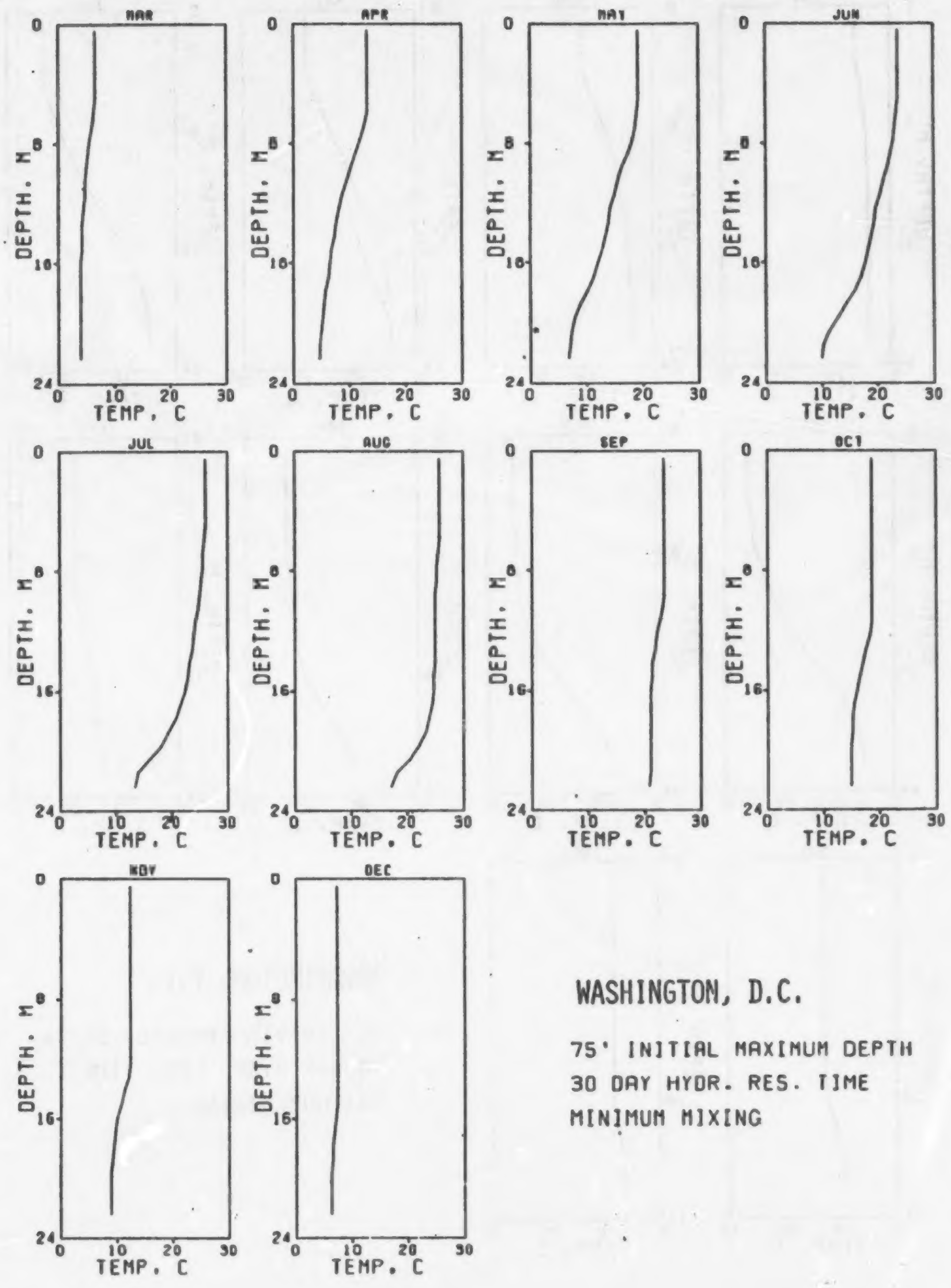
WASHINGTON, D.C.

75' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



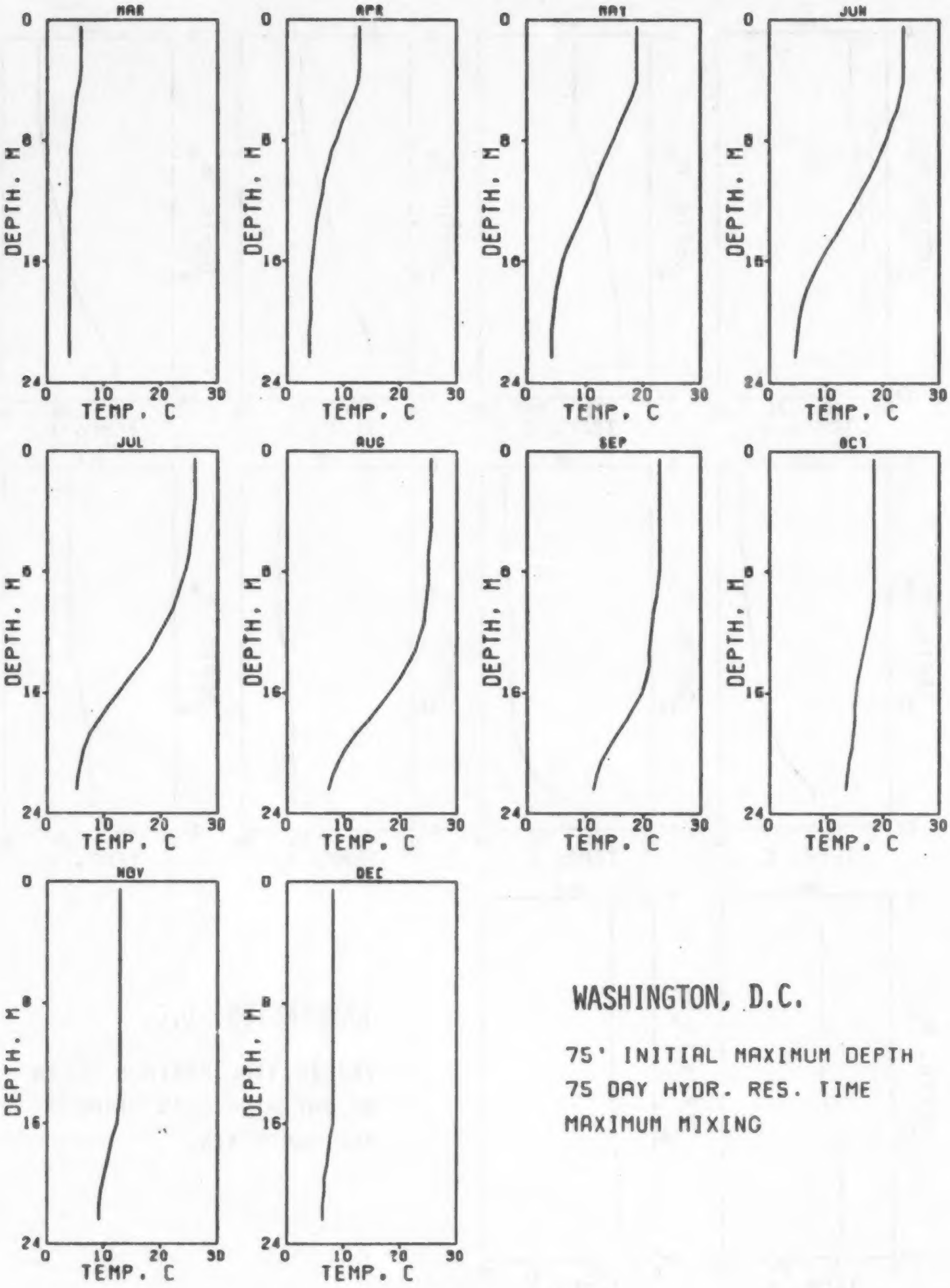
WASHINGTON, D.C.

75' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



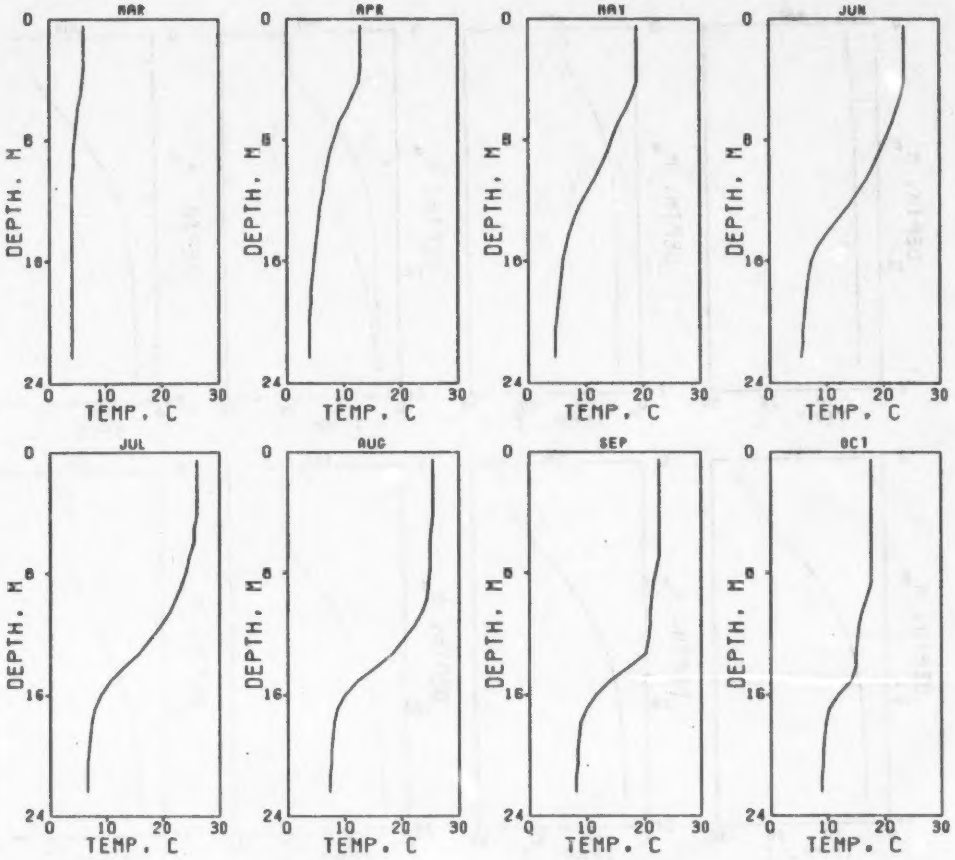
WASHINGTON, D.C.

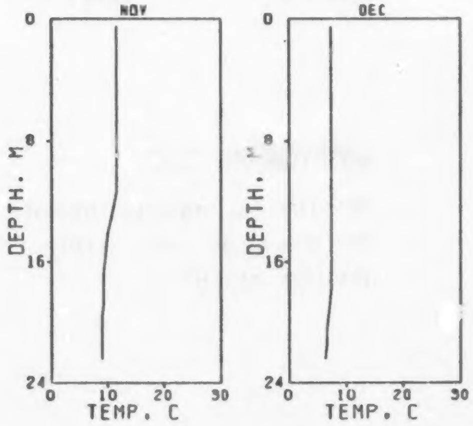
75' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



WASHINGTON, D.C.

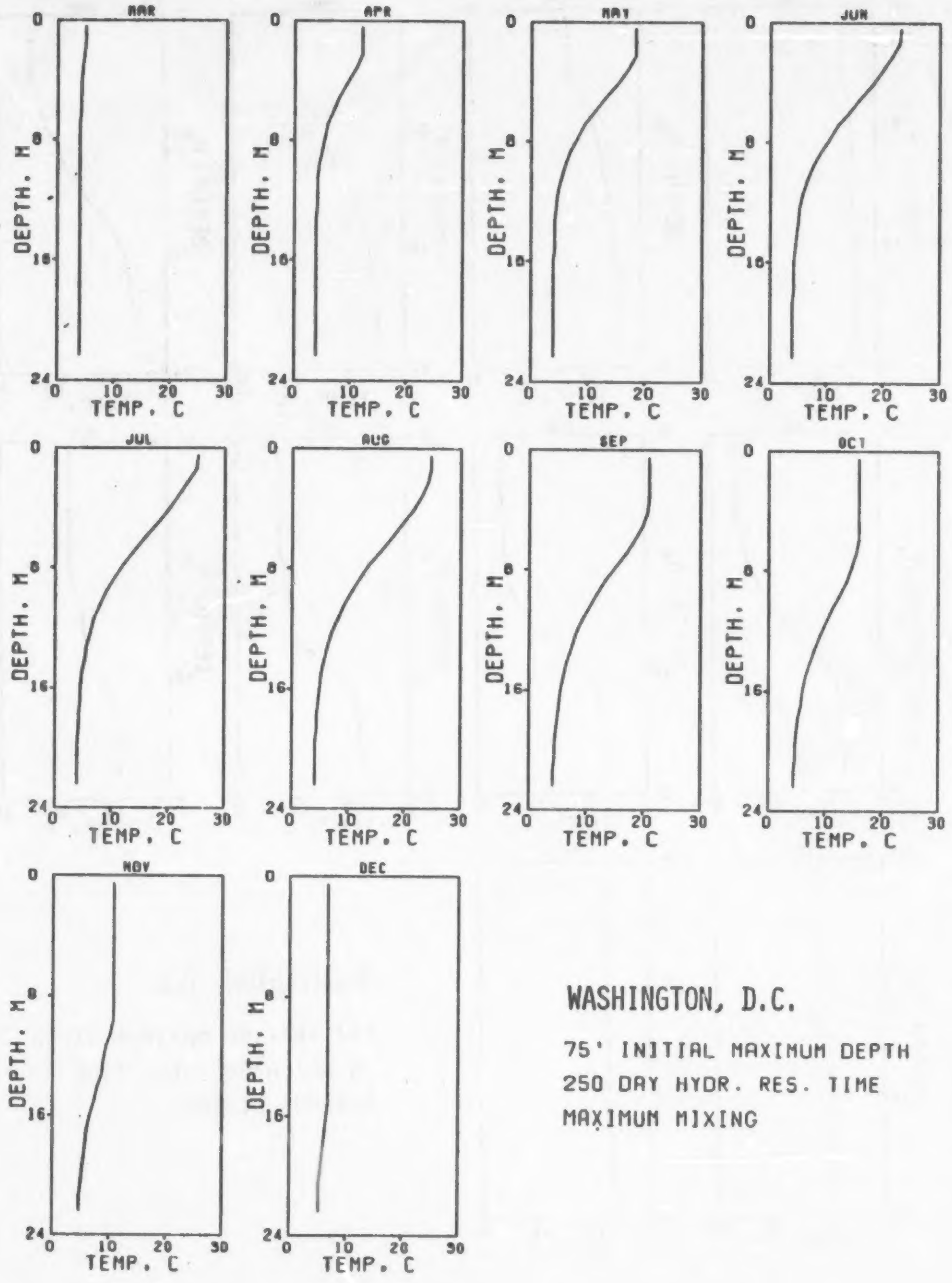
75' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING





WASHINGTON, D.C.

75' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING

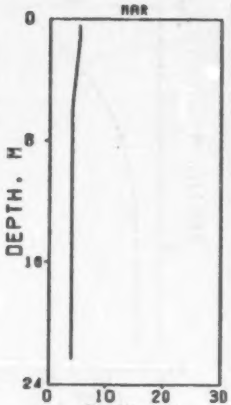


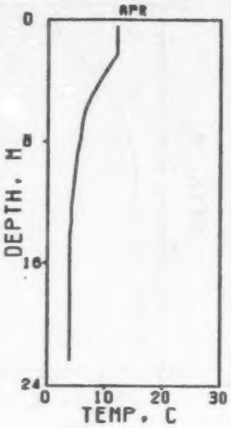
WASHINGTON, D.C.

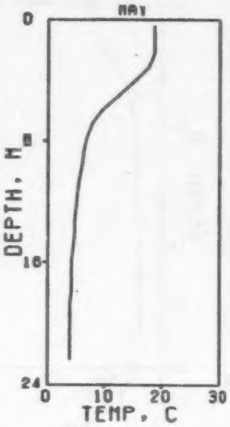
75' INITIAL MAXIMUM DEPTH

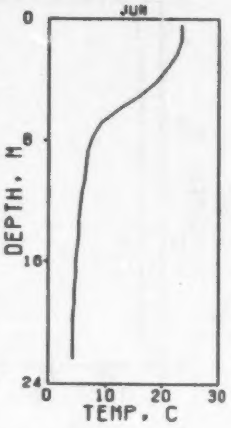
250 DAY HYDR. RES. TIME

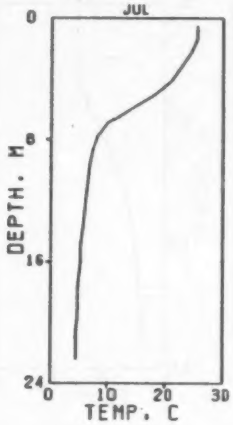
MAXIMUM MIXING

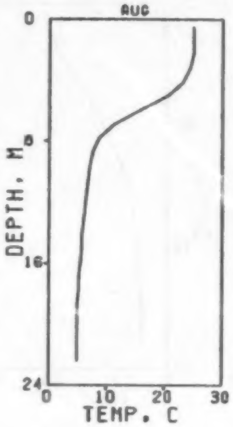


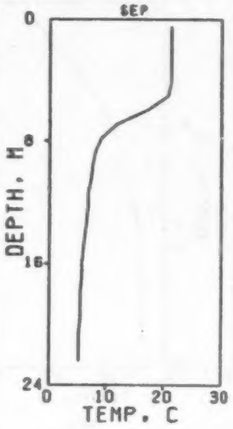


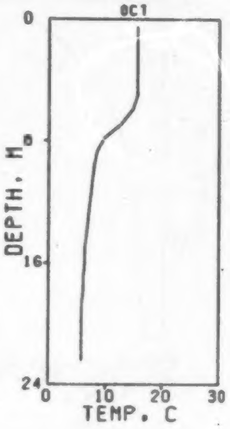


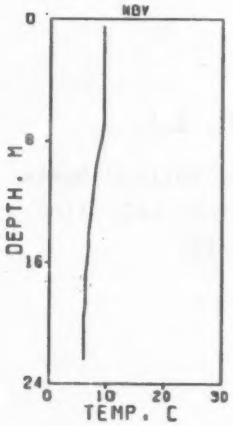


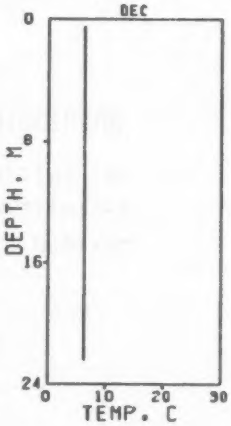






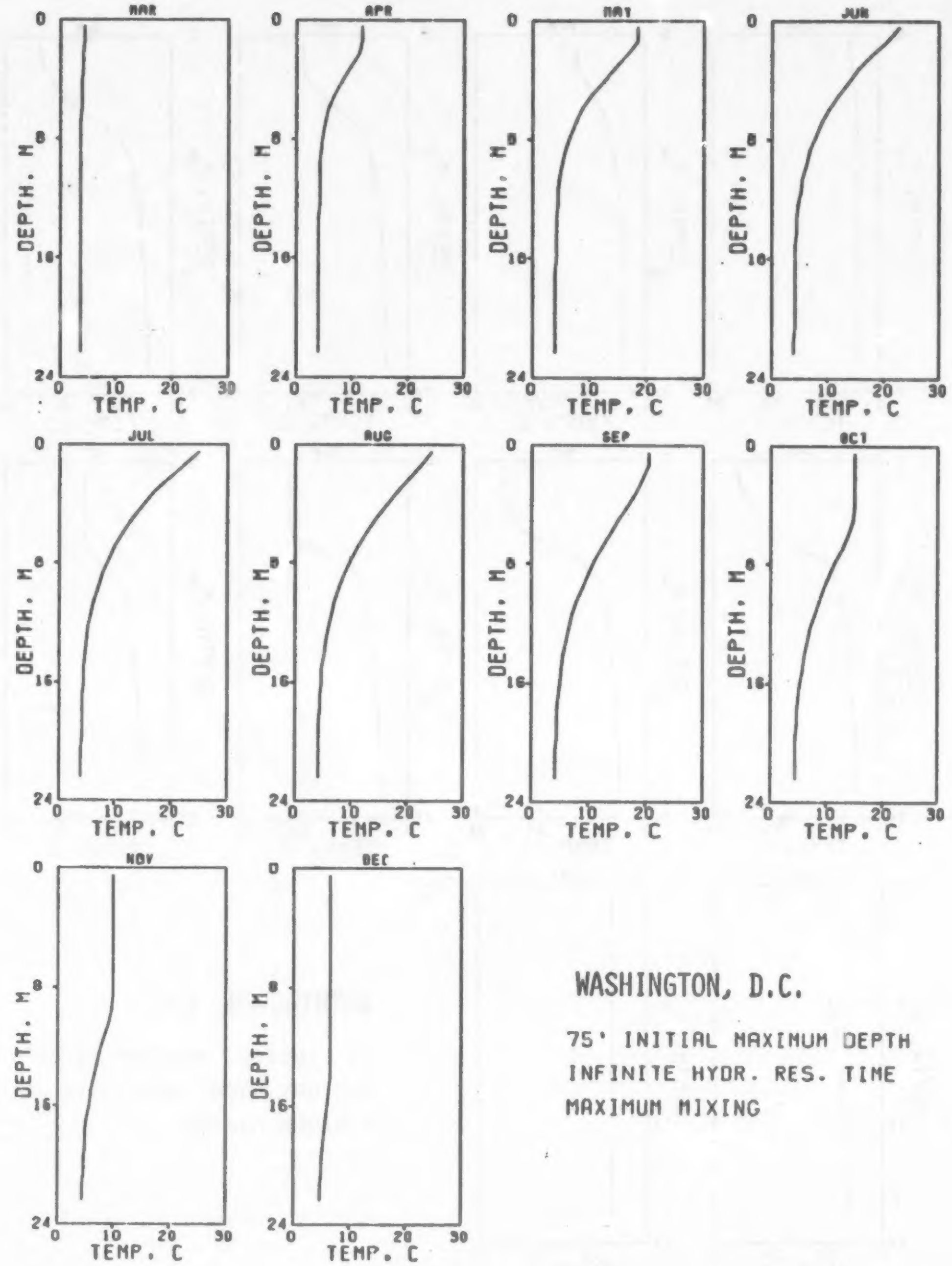






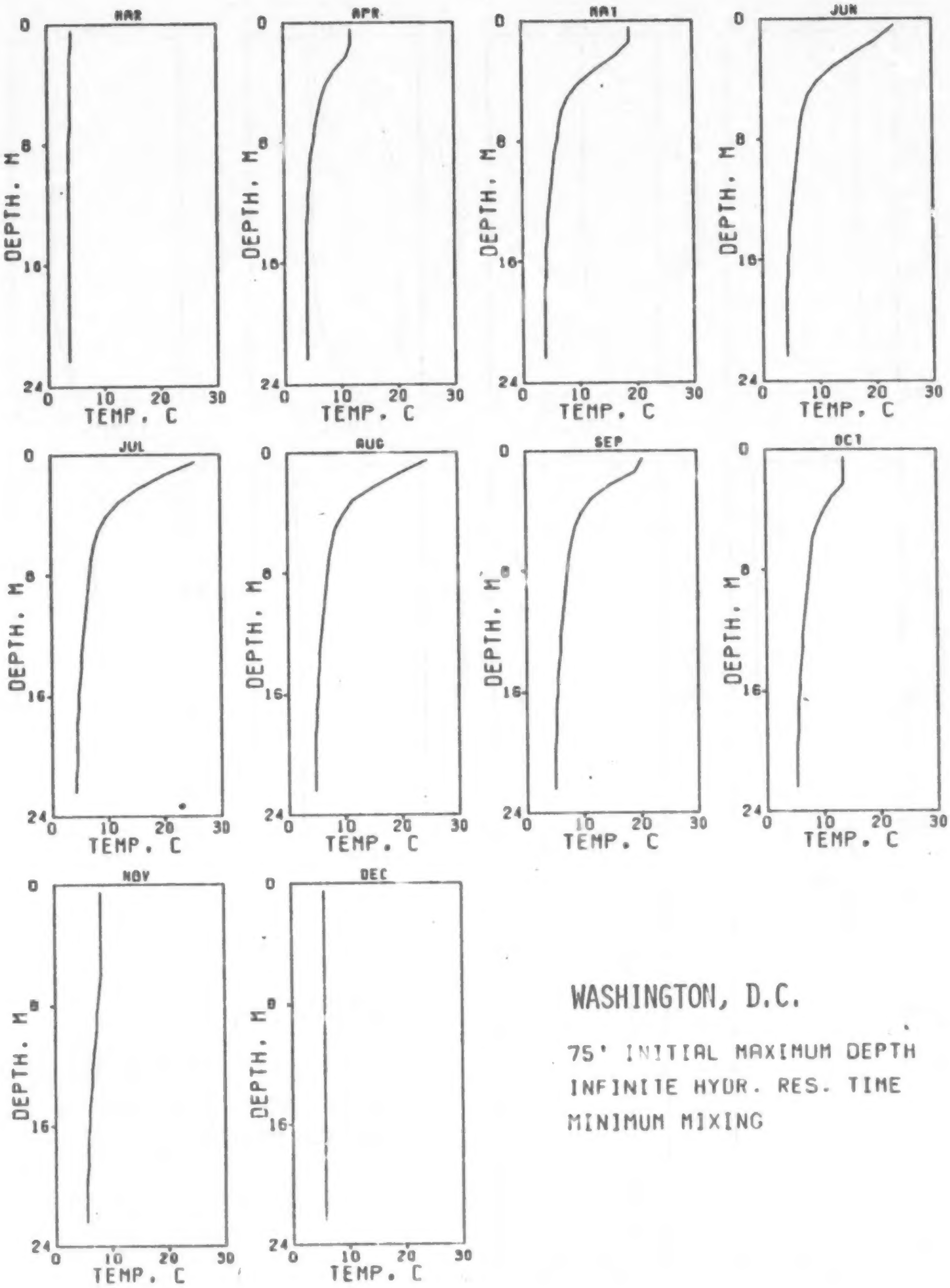
WASHINGTON, D.C.

75' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



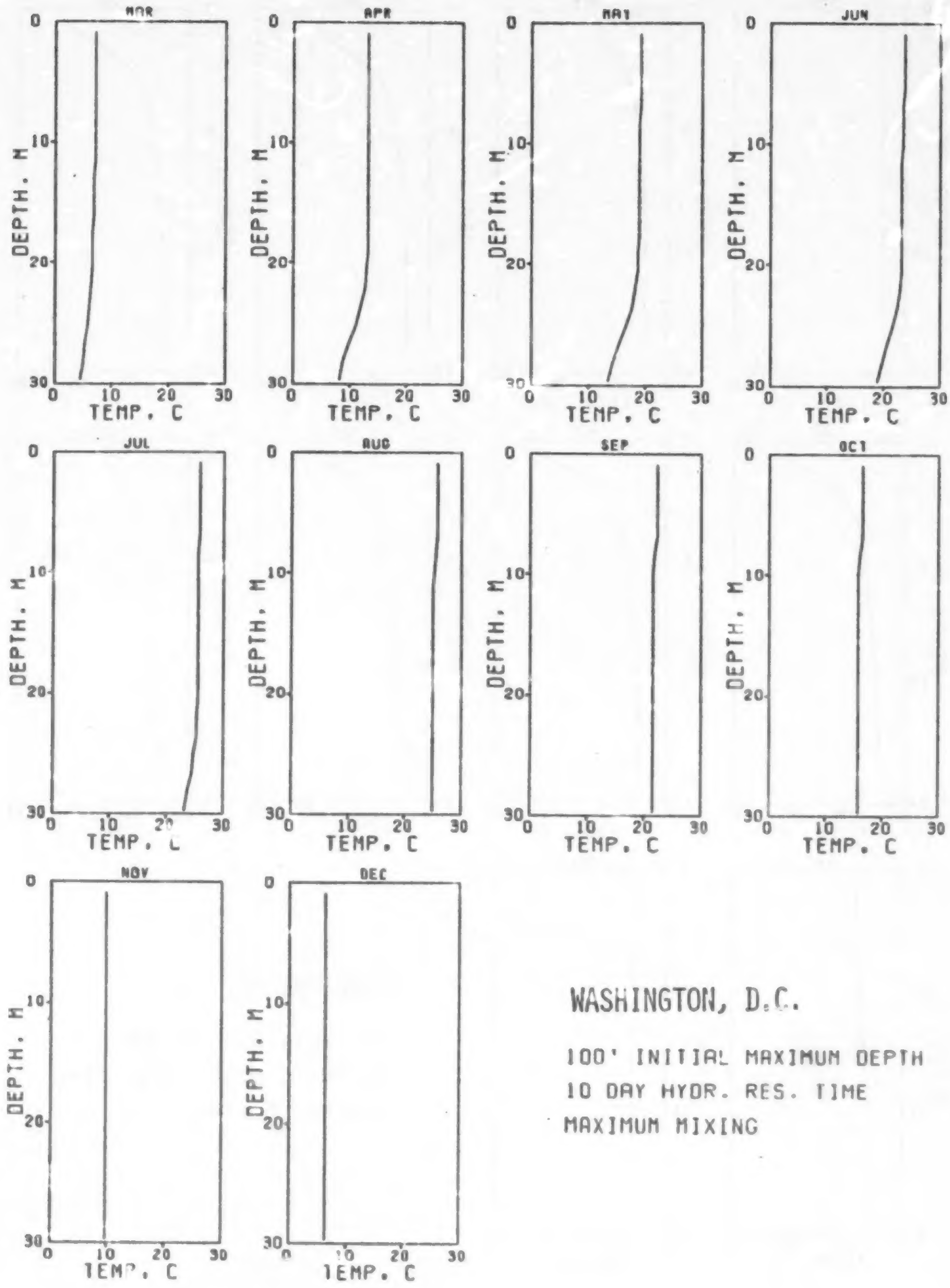
WASHINGTON, D.C.

75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



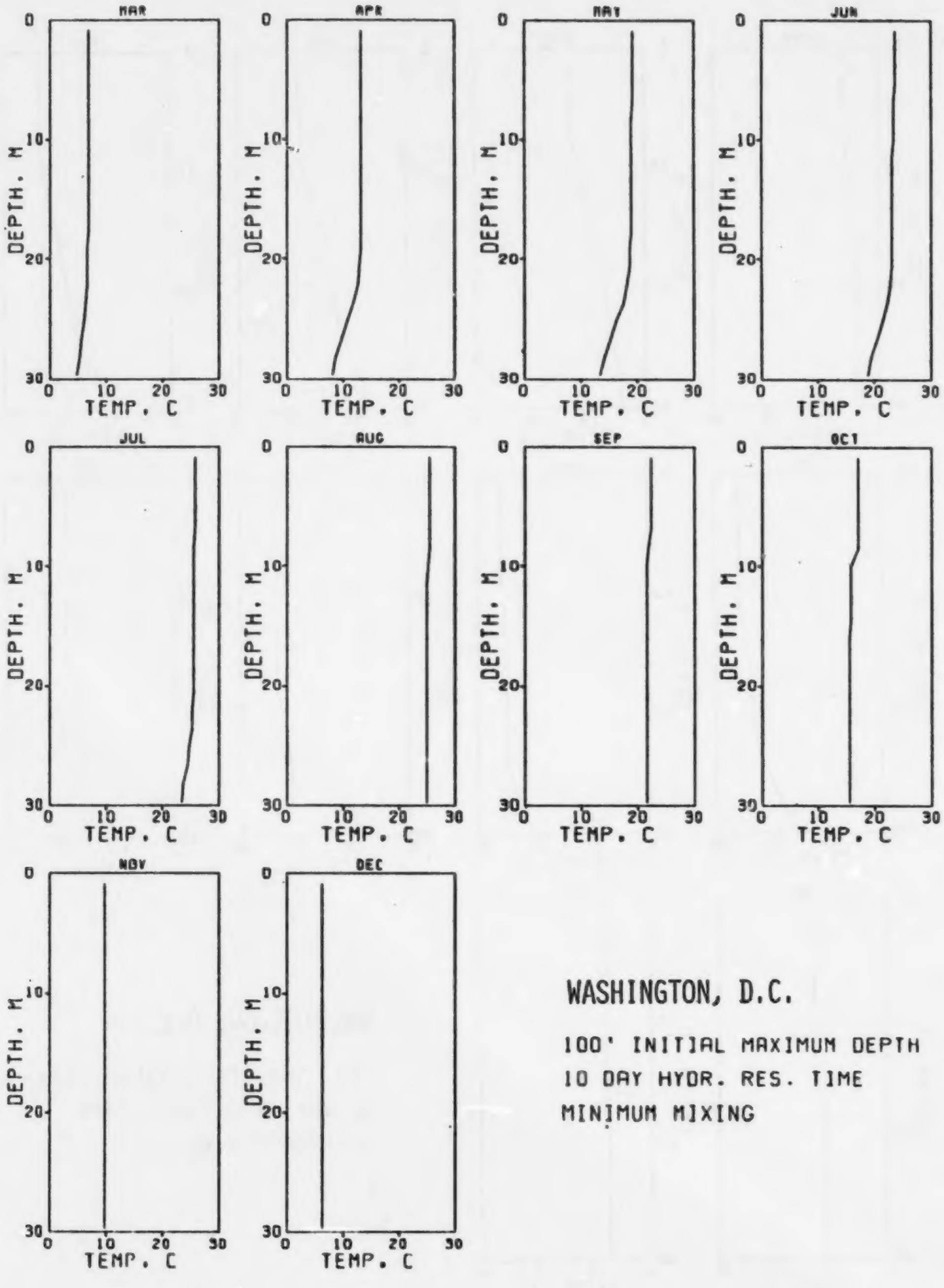
WASHINGTON, D.C.

75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



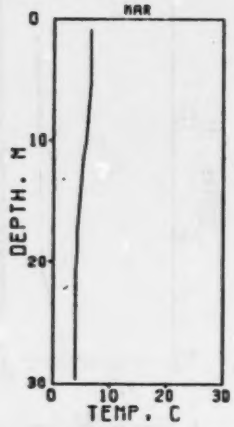
WASHINGTON, D.C.

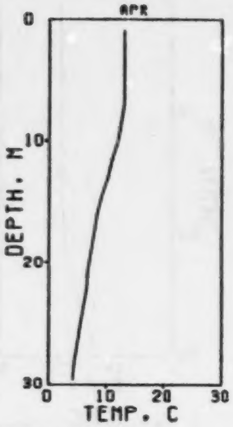
100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING

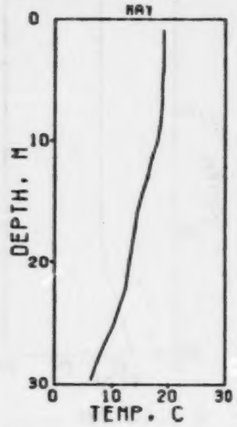


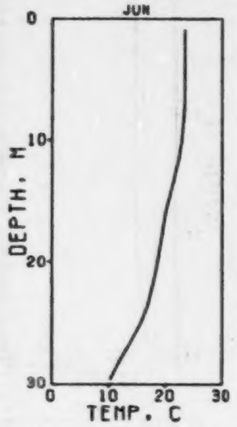
WASHINGTON, D.C.

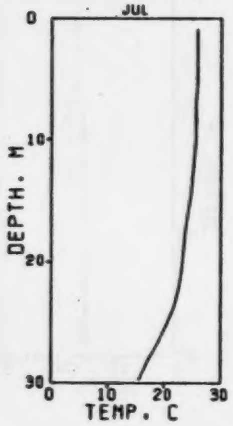
100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING

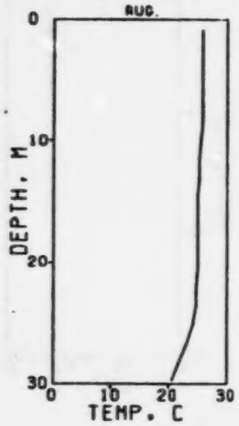


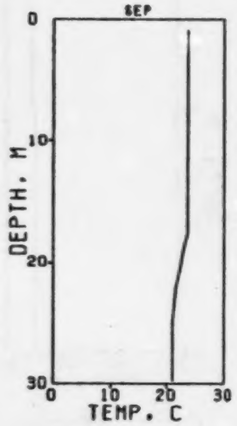


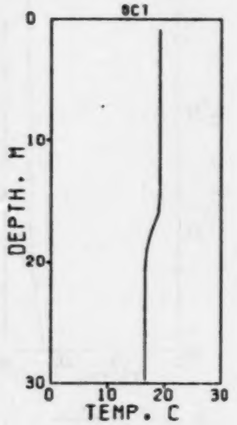


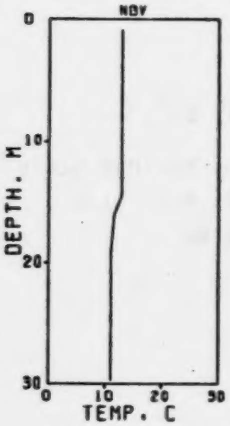


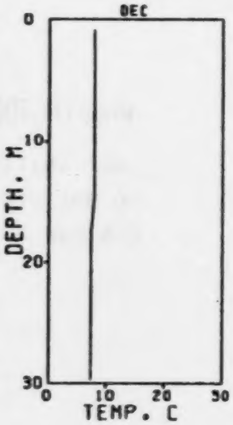






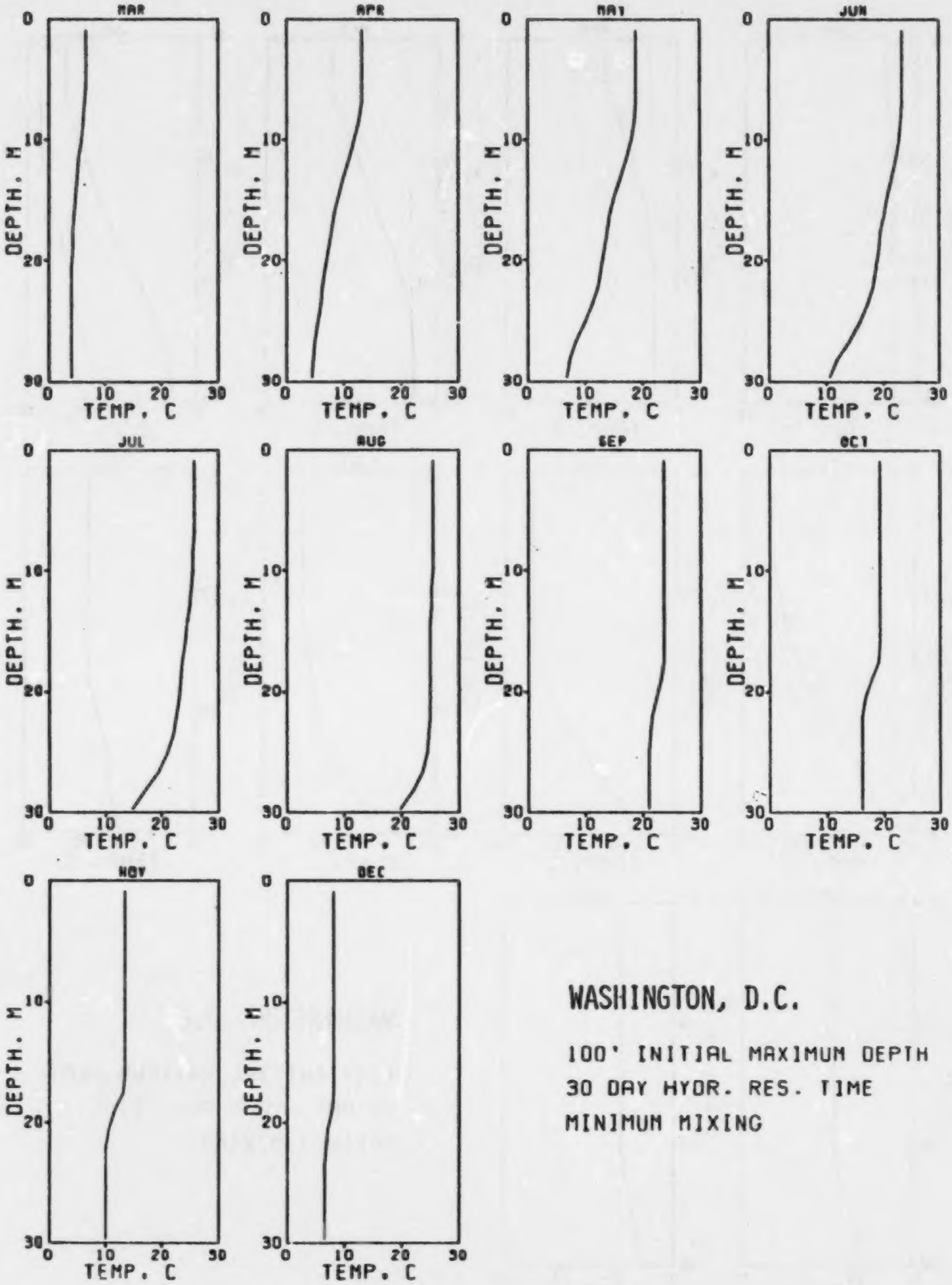






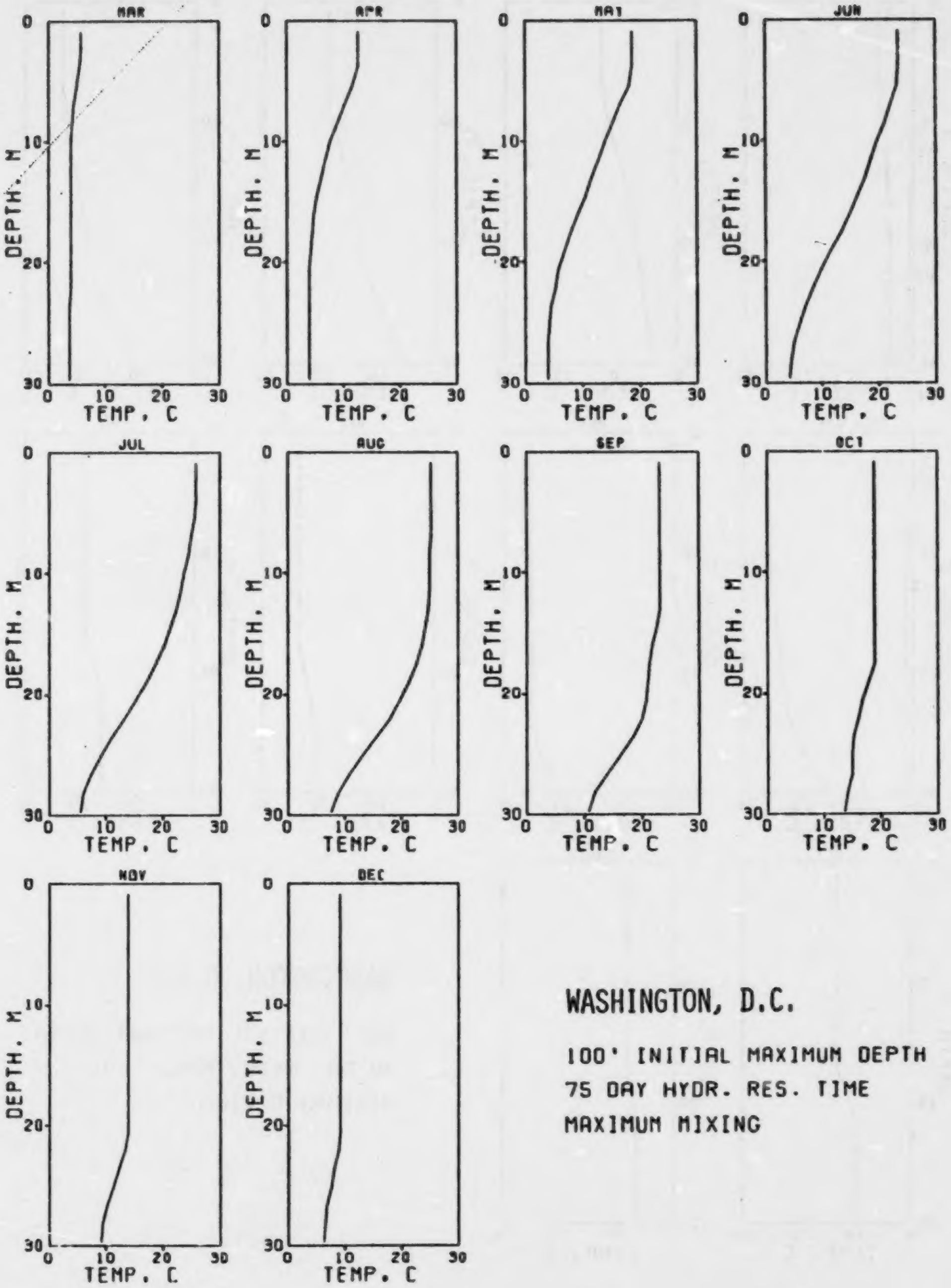
WASHINGTON, D.C.

100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



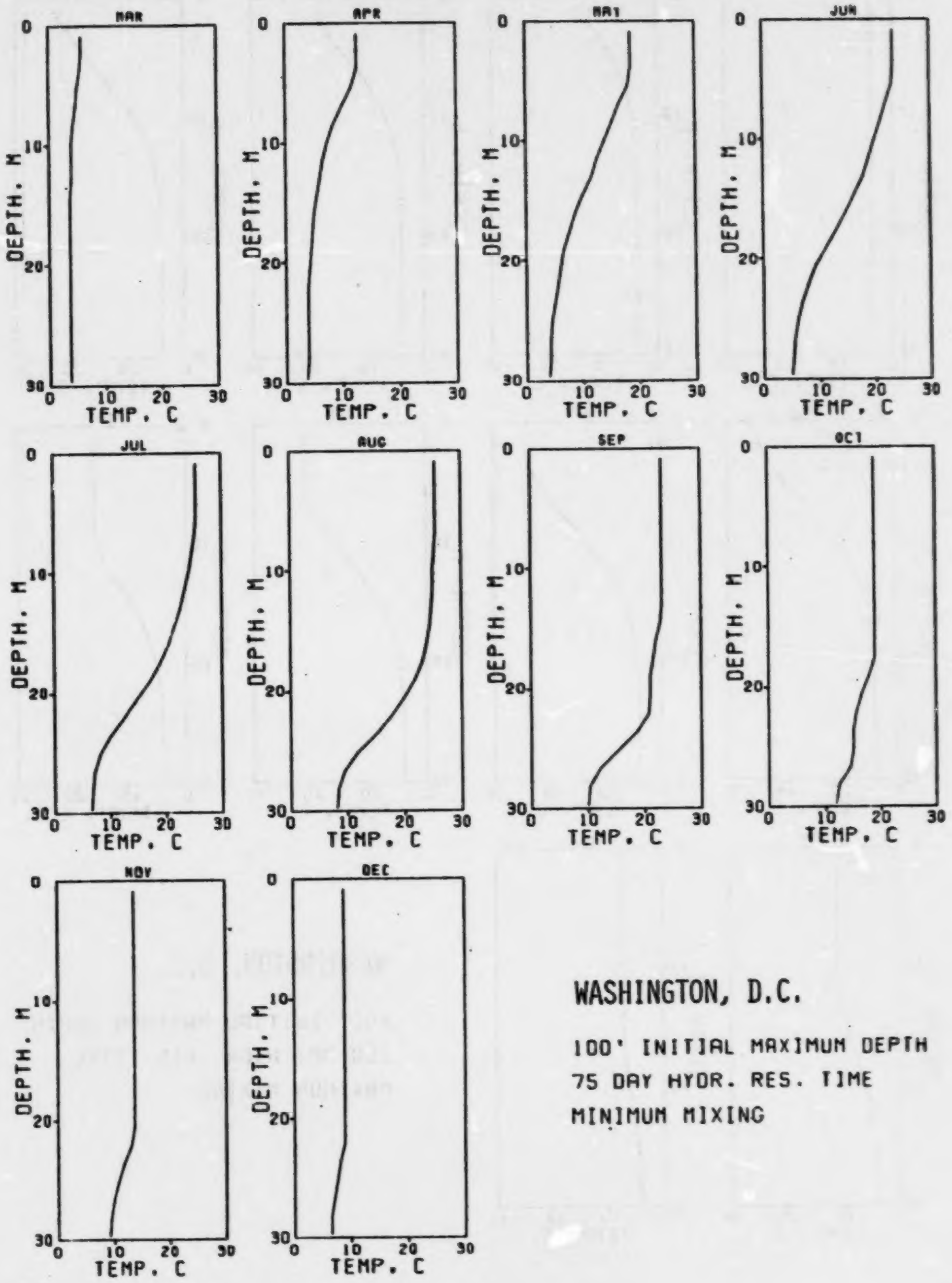
WASHINGTON, D.C.

100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



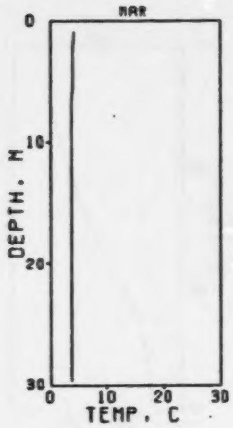
WASHINGTON, D.C.

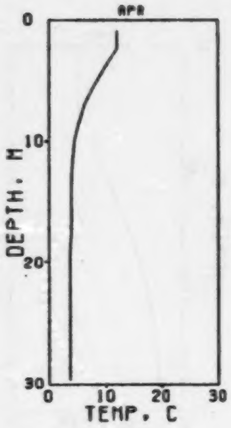
100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING

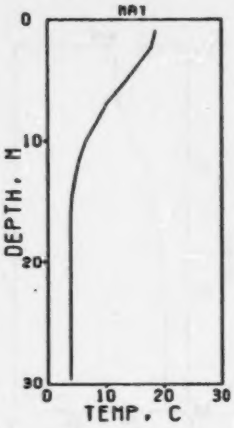


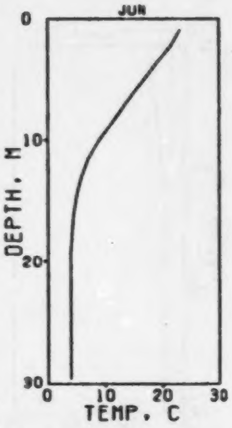
WASHINGTON, D.C.

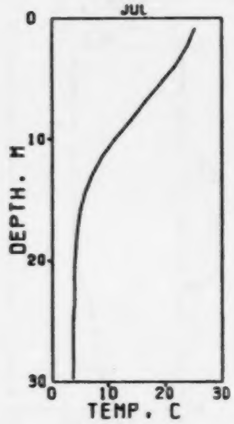
100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING

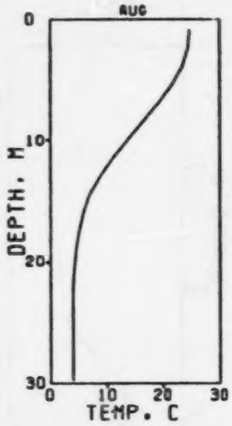


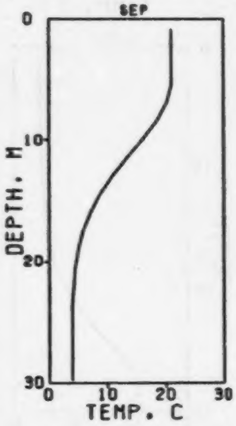


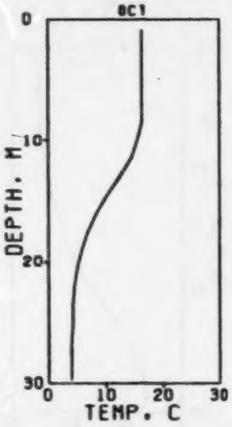


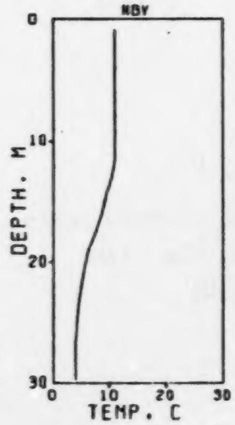


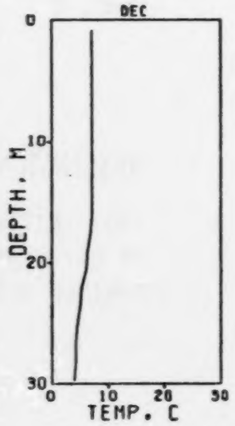






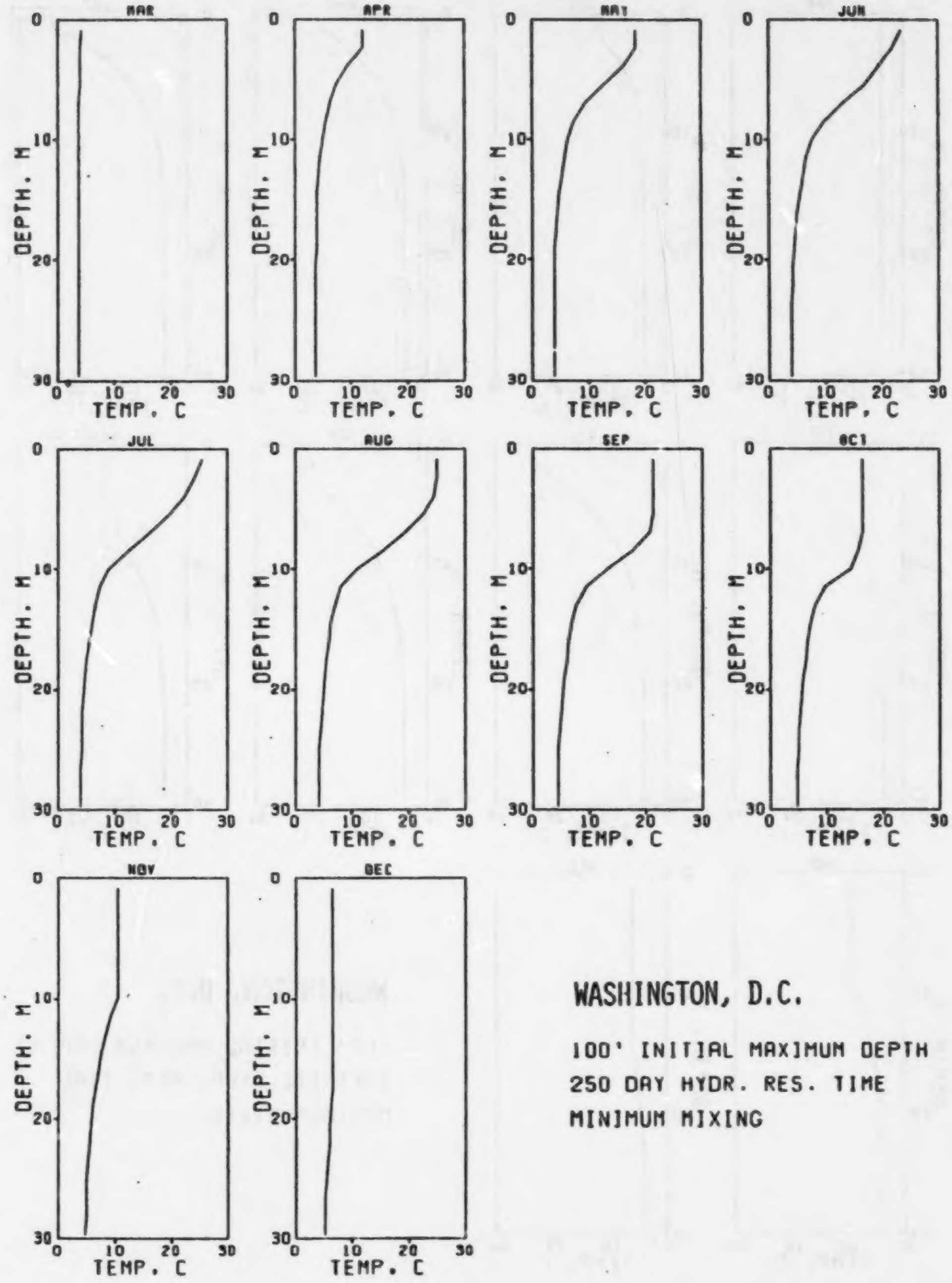






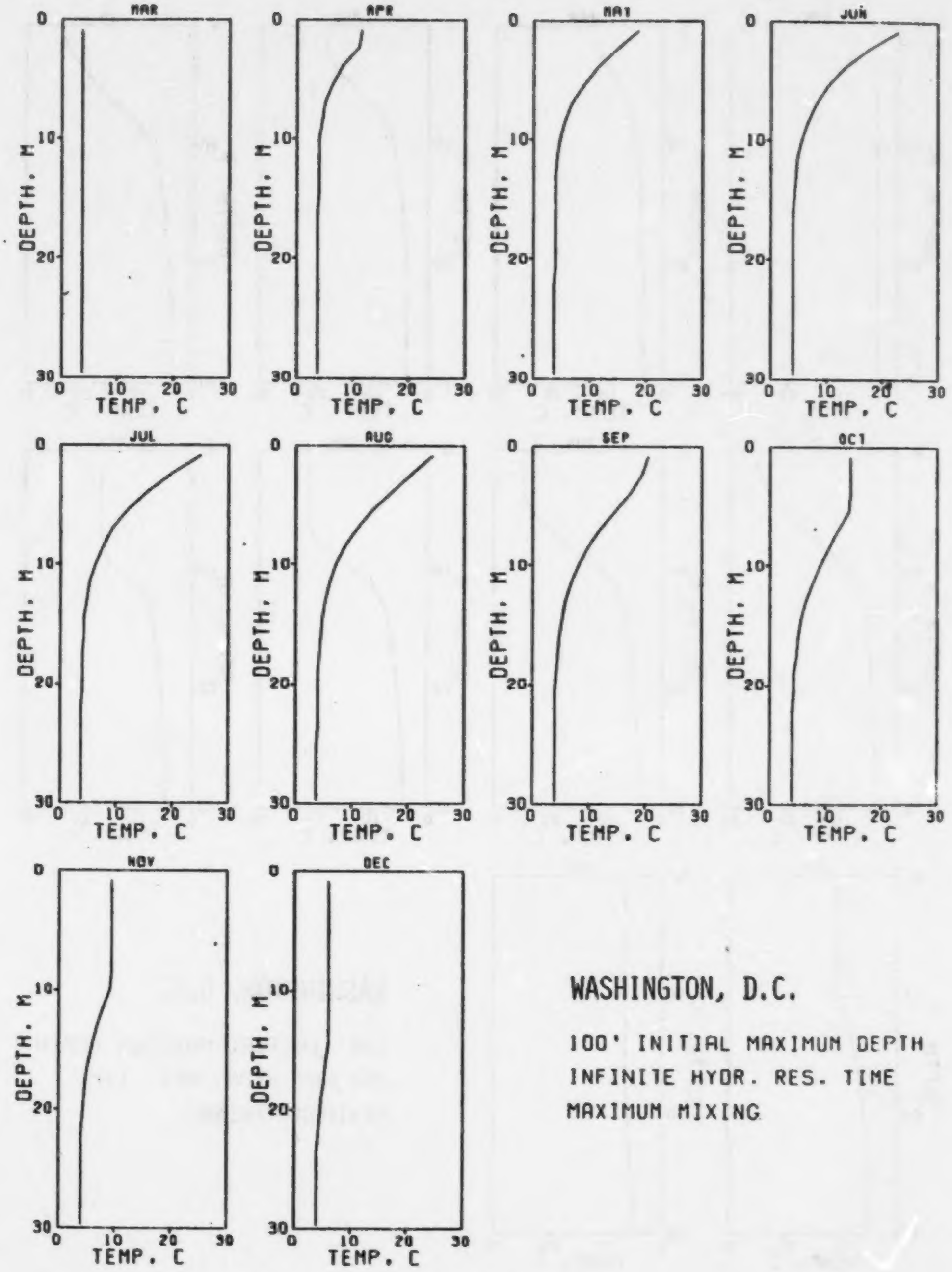
WASHINGTON, D.C.

100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



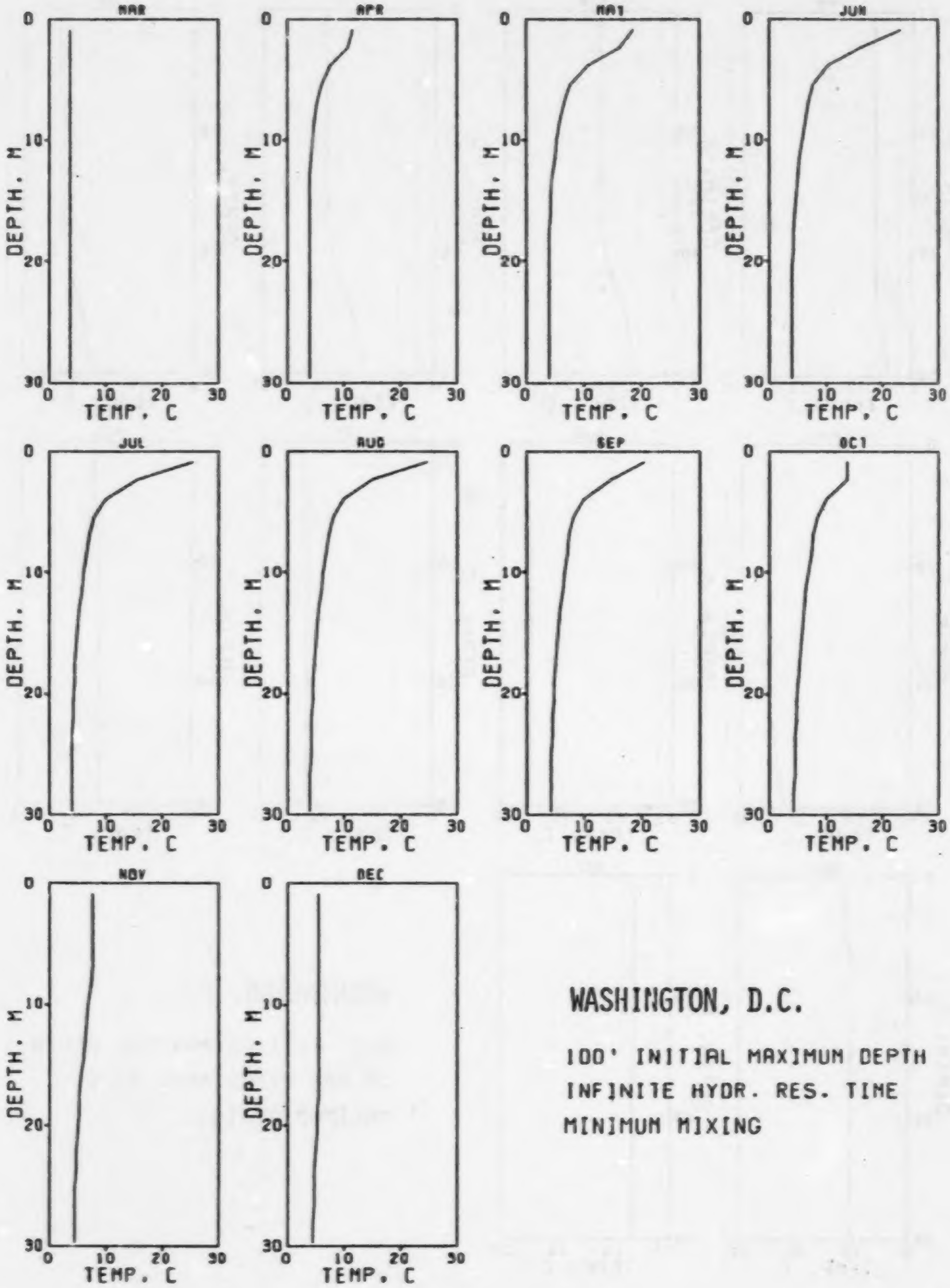
WASHINGTON, D.C.

100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



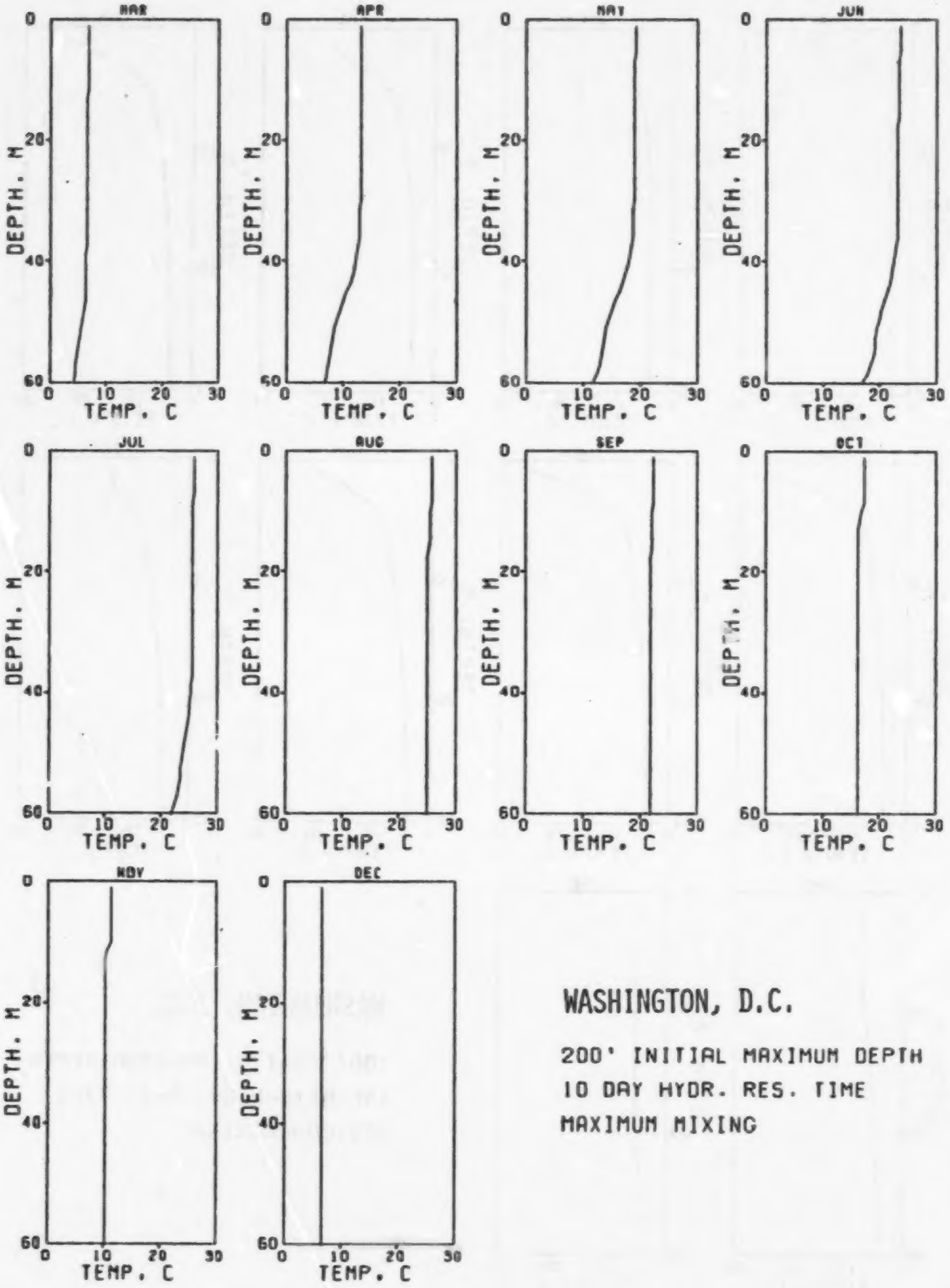
WASHINGTON, D.C.

100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



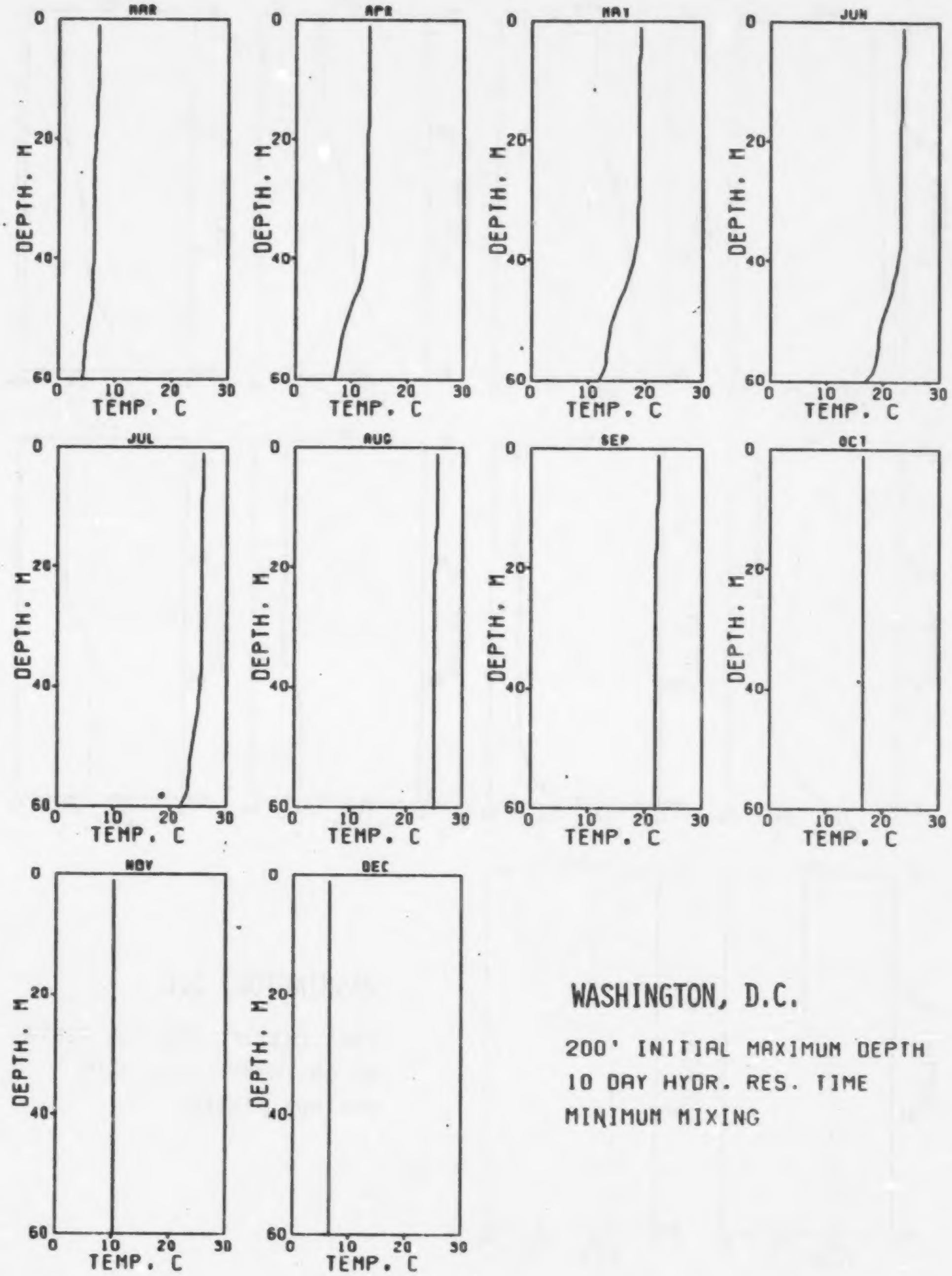
WASHINGTON, D.C.

100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



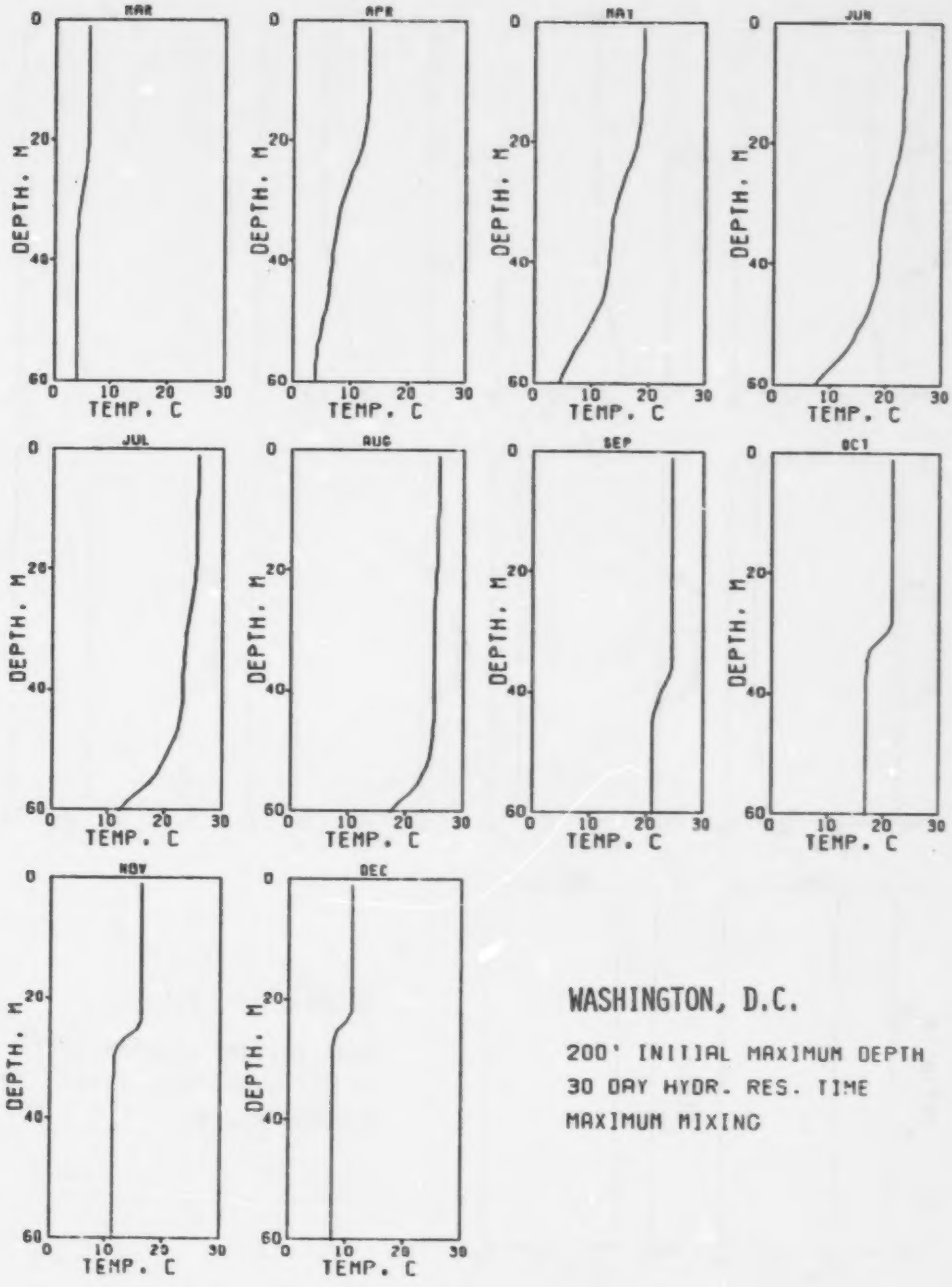
WASHINGTON, D.C.

200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



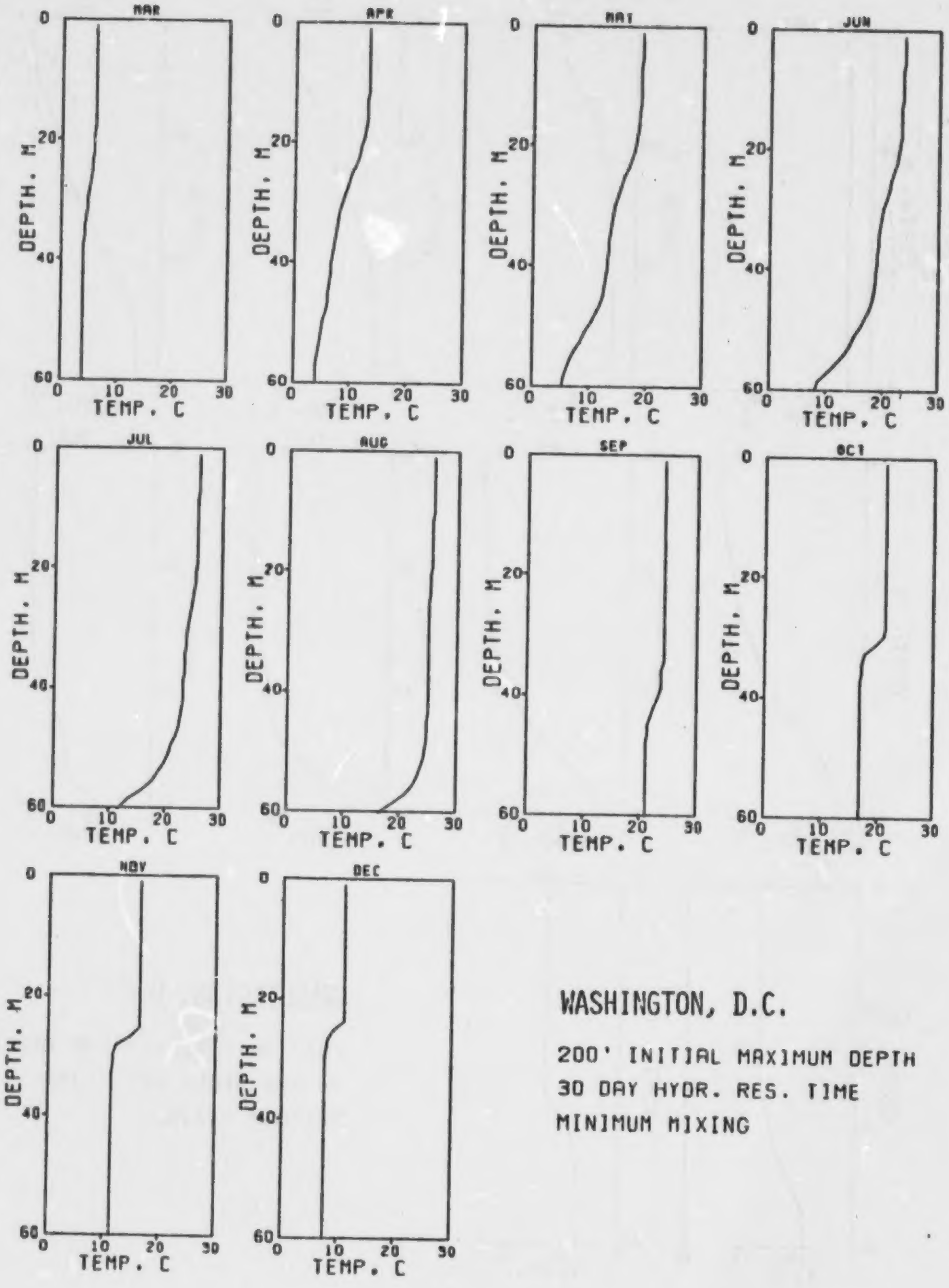
WASHINGTON, D.C.

200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



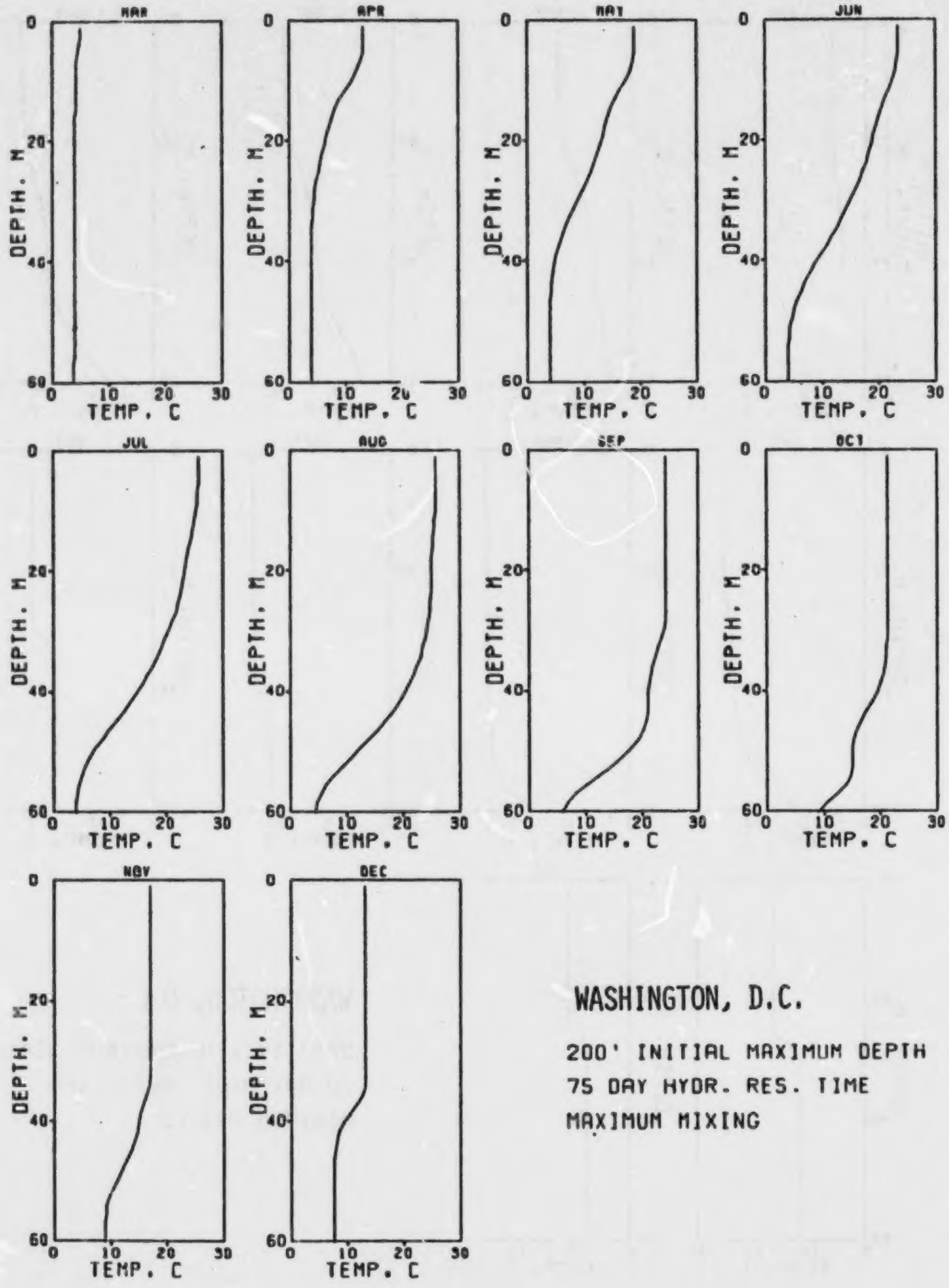
WASHINGTON, D.C.

200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



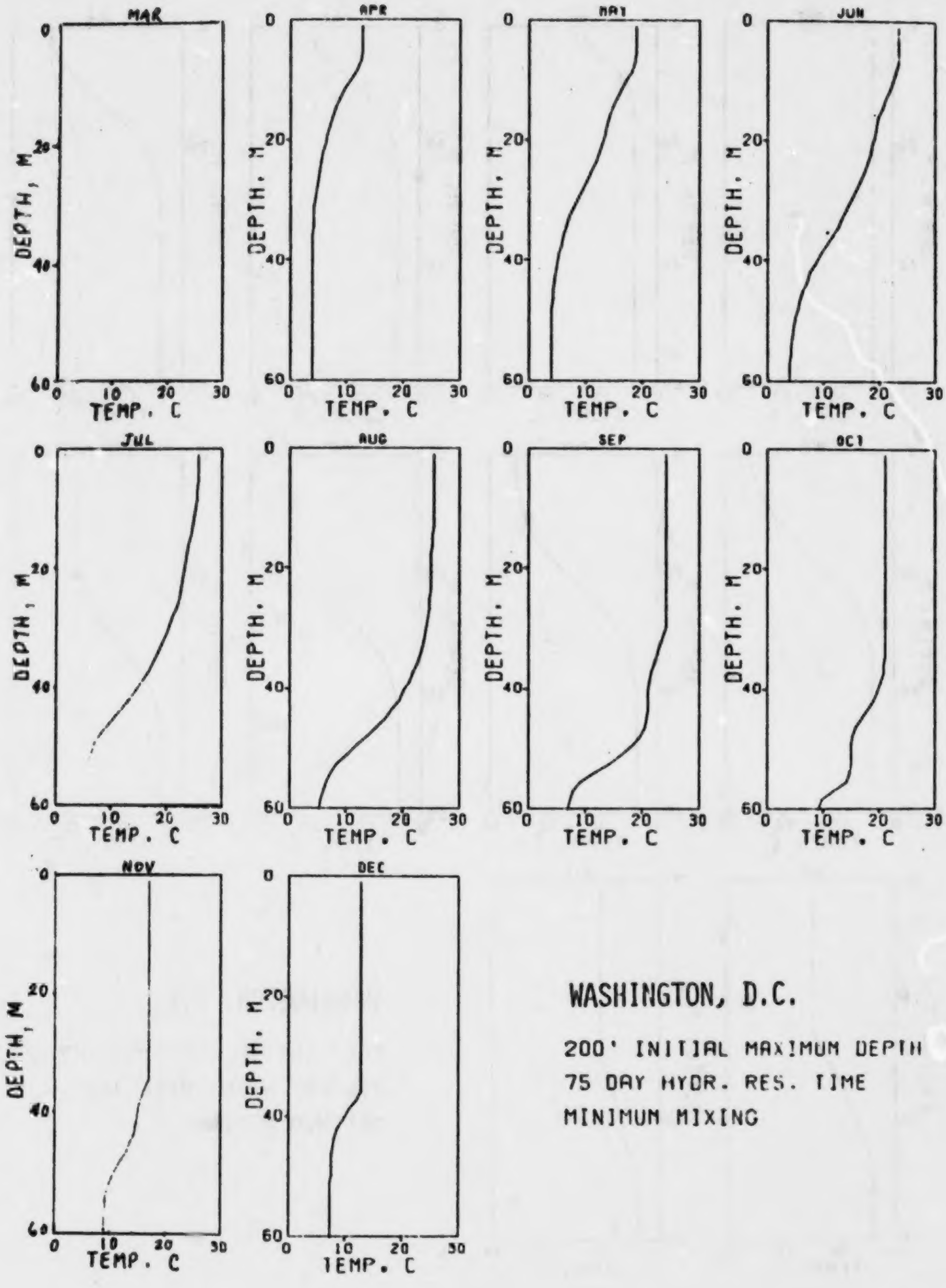
WASHINGTON, D.C.

200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



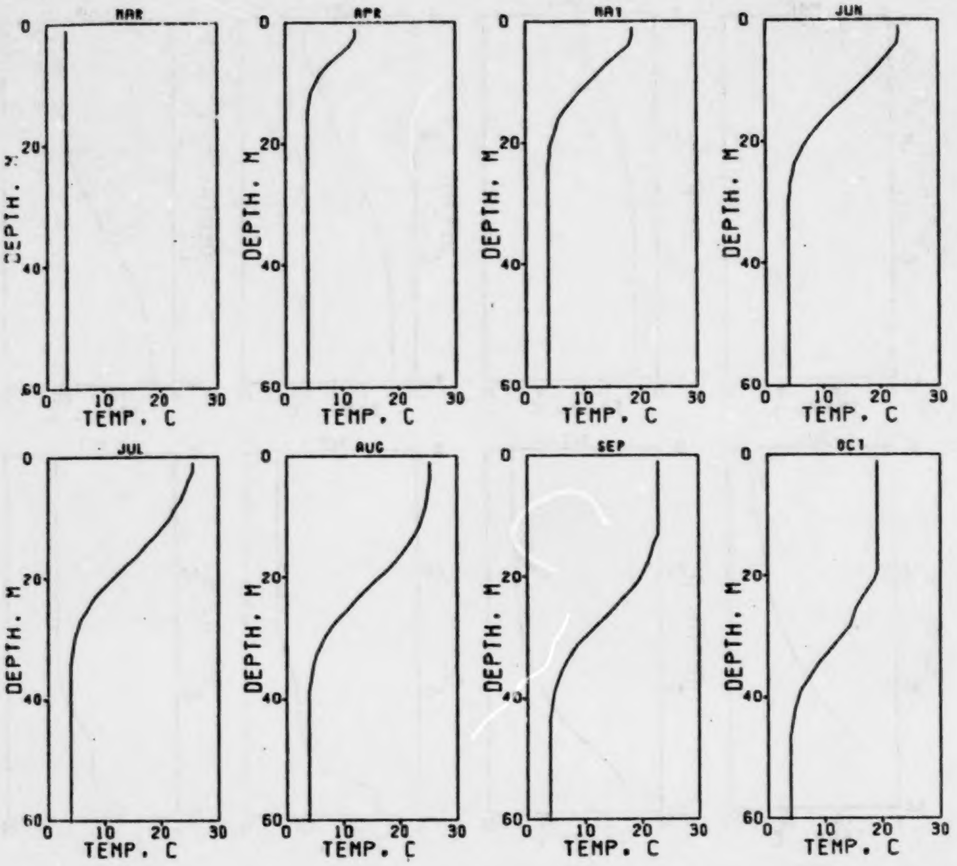
WASHINGTON, D.C.

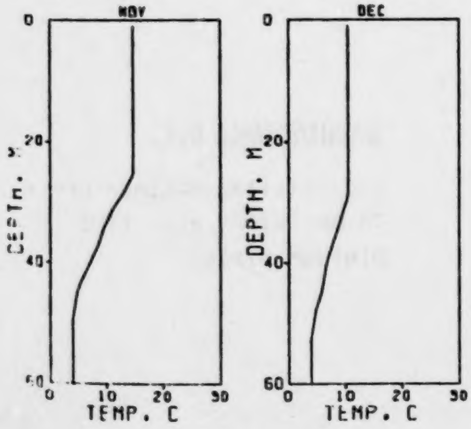
200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



WASHINGTON, D.C.

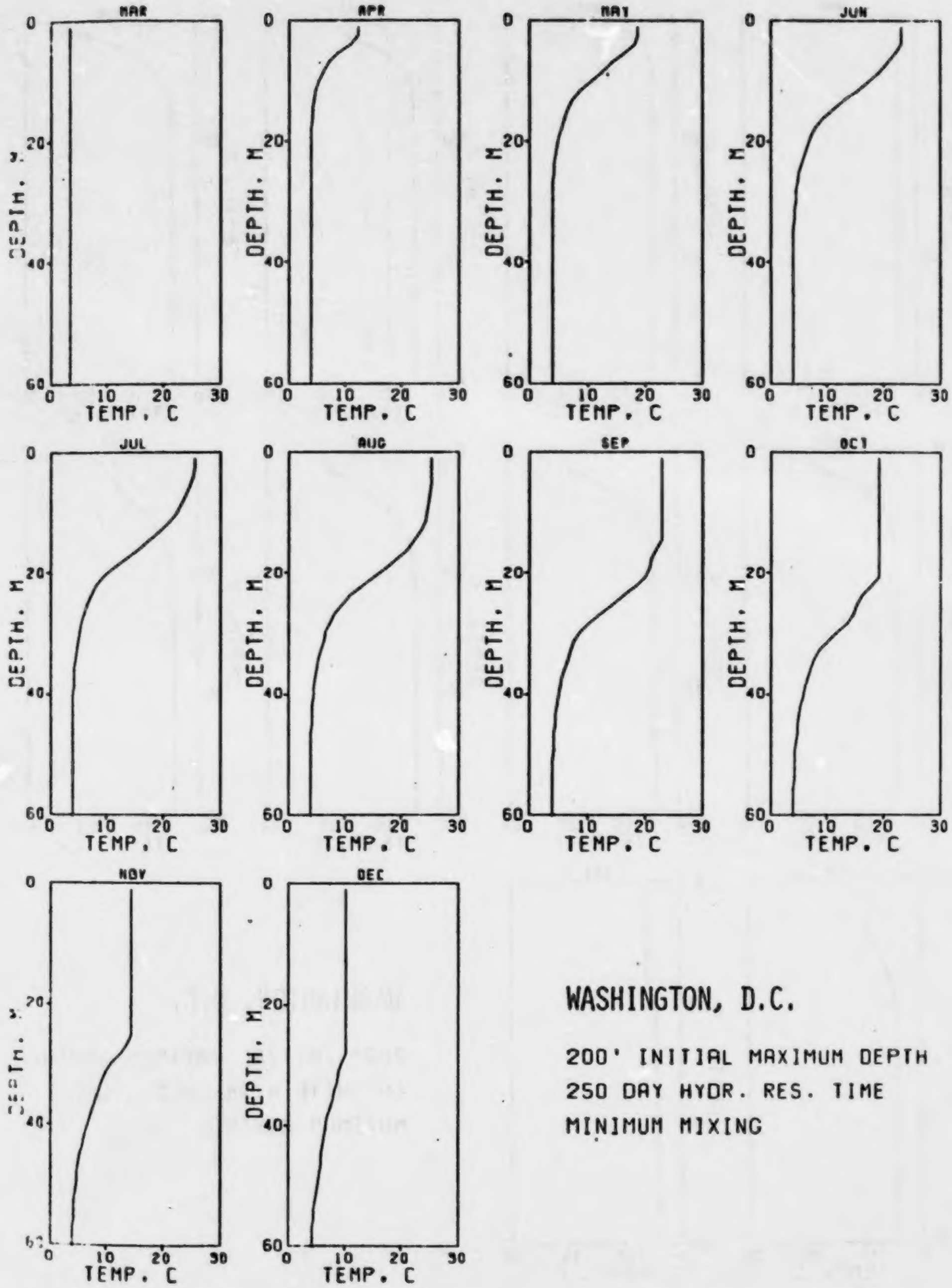
200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING





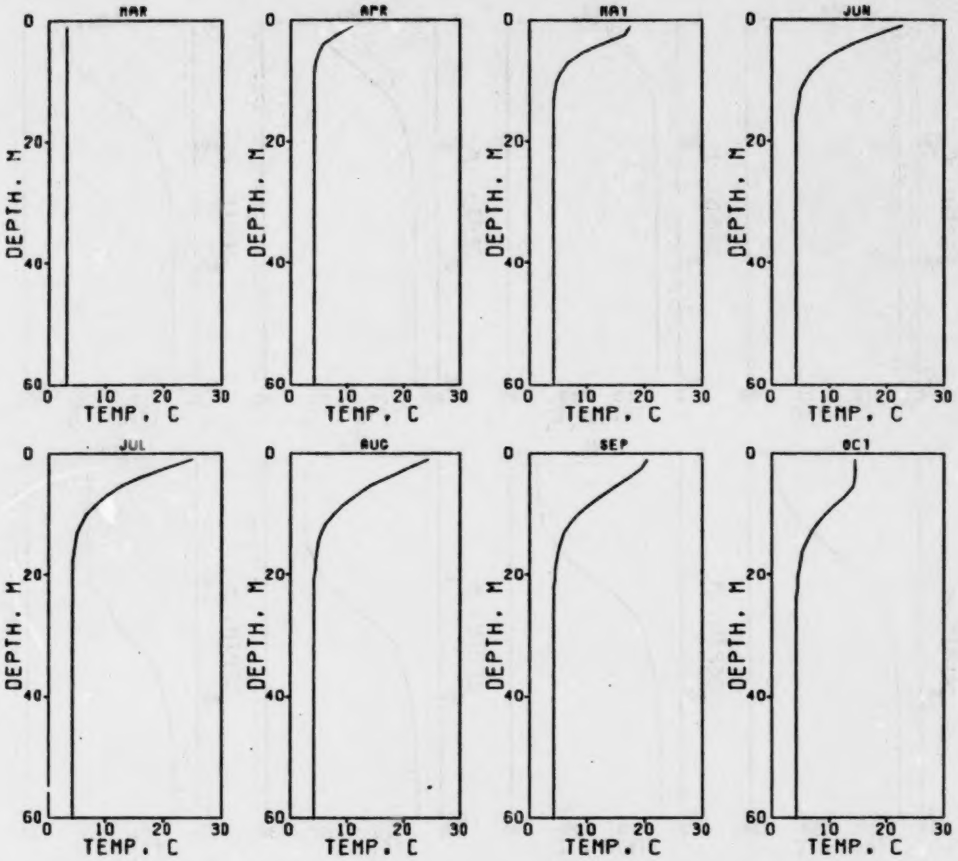
WASHINGTON, D.C.

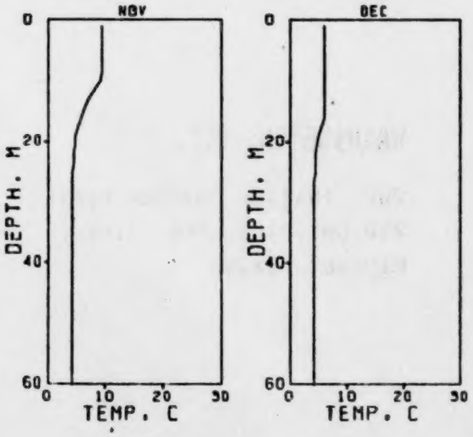
200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



WASHINGTON, D.C.

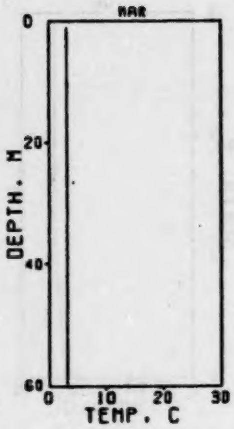
200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING

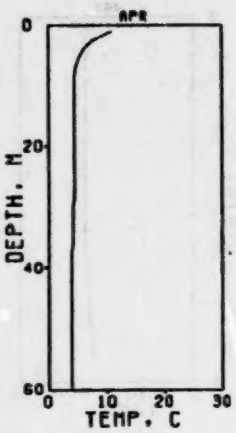


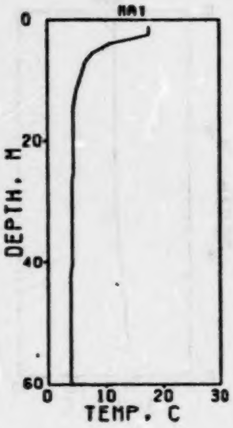


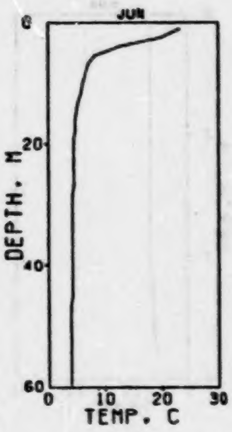
WASHINGTON, D.C.

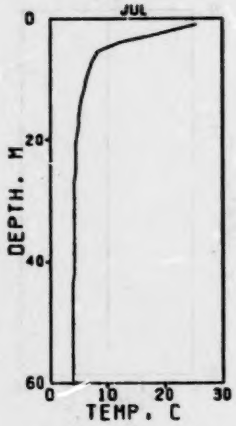
200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING

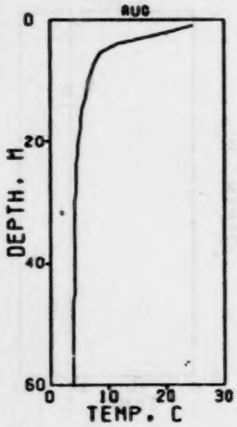


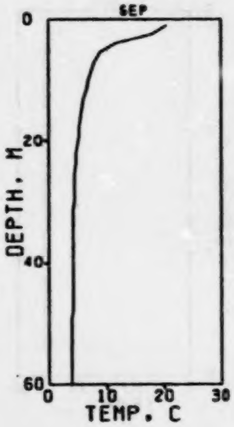


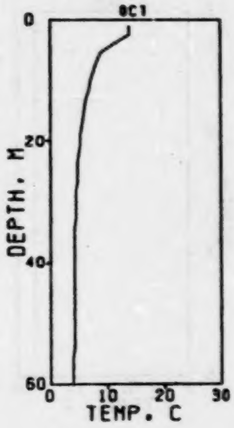


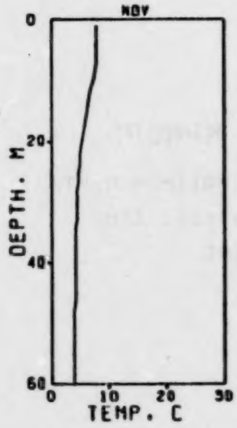


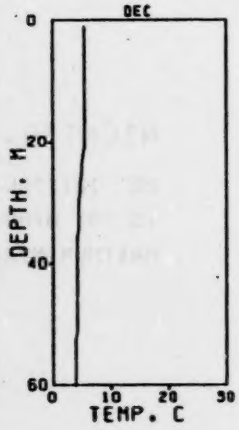






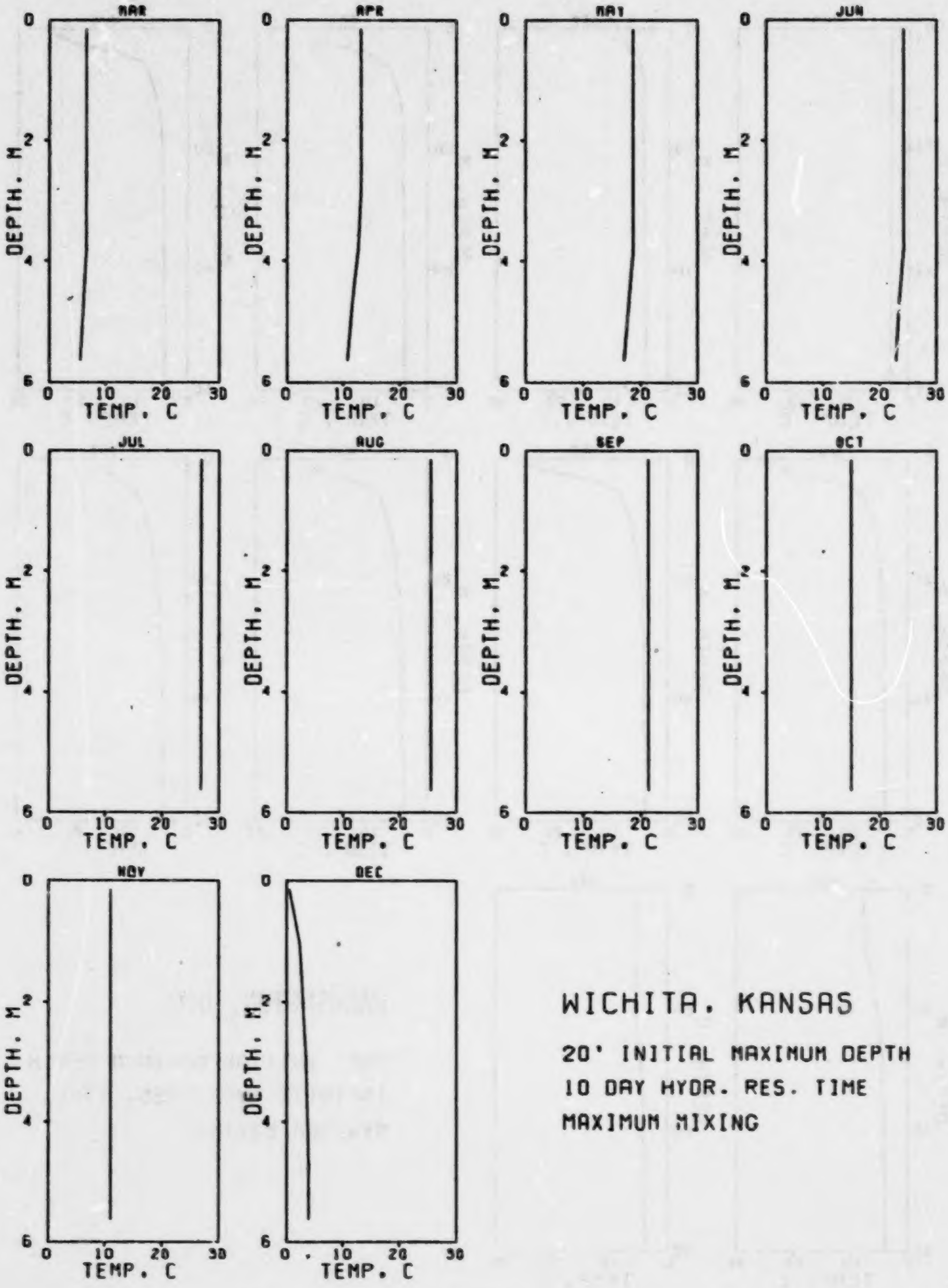






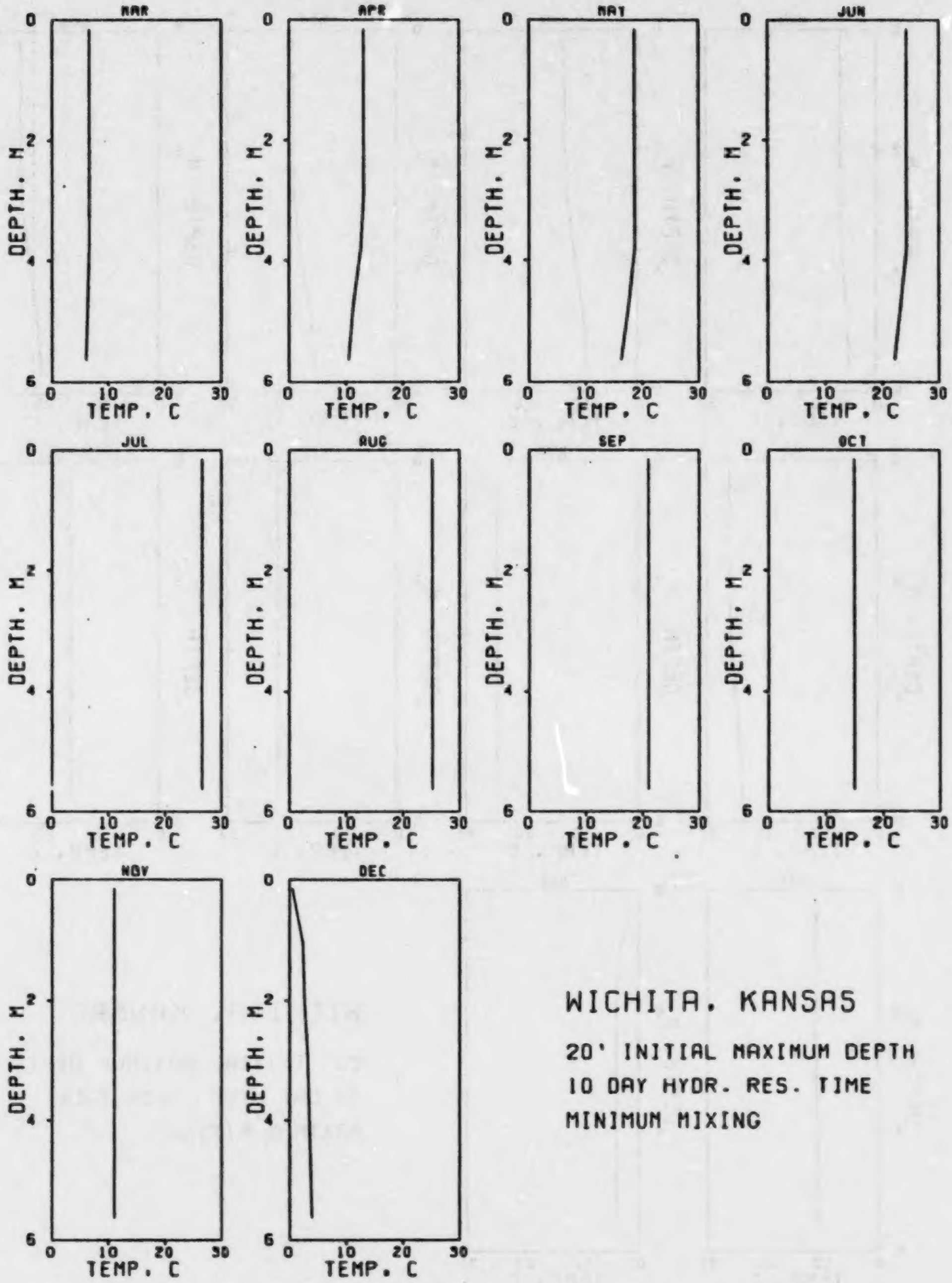
WASHINGTON, D.C.

200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



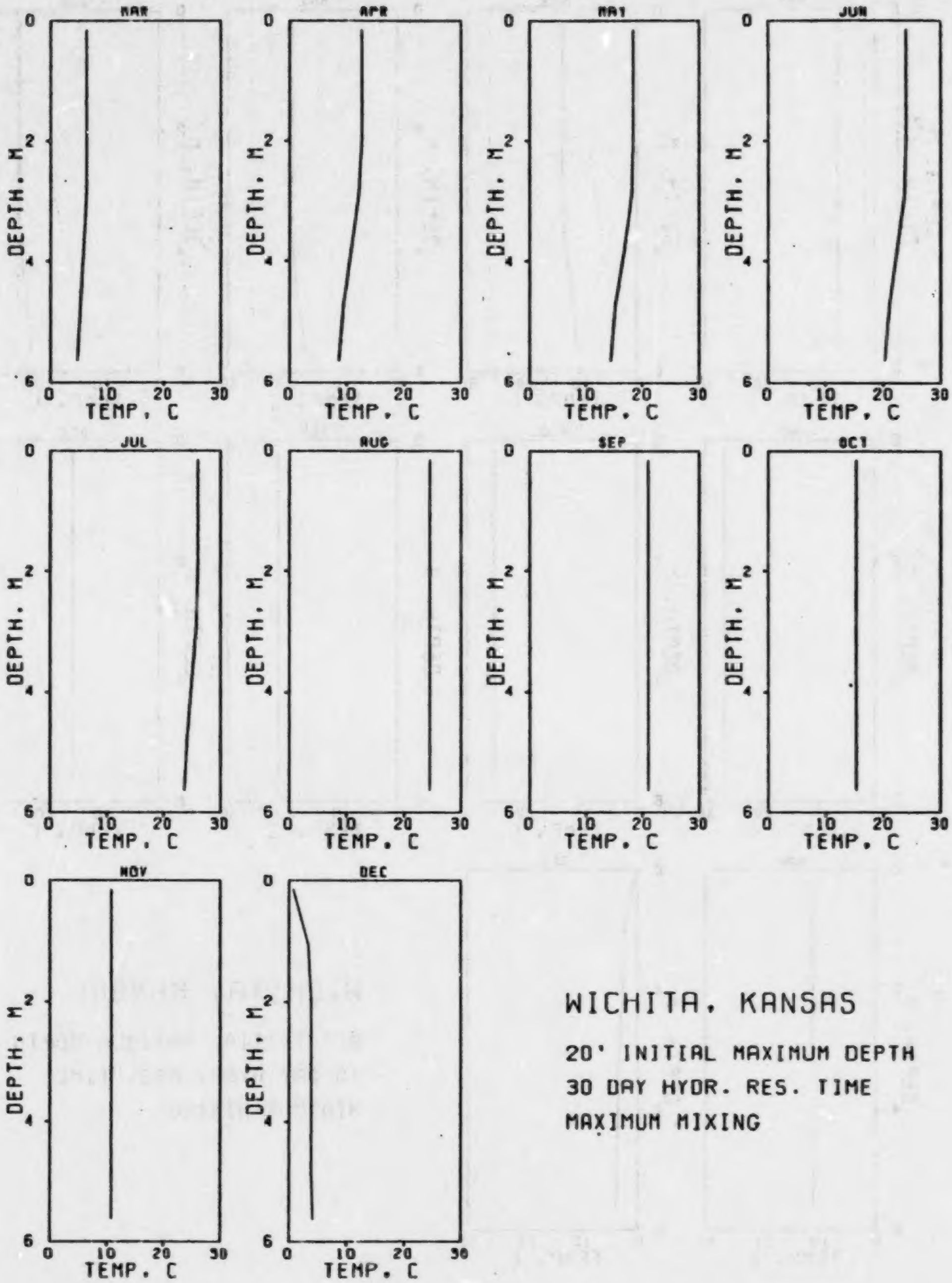
WICHITA, KANSAS

20' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



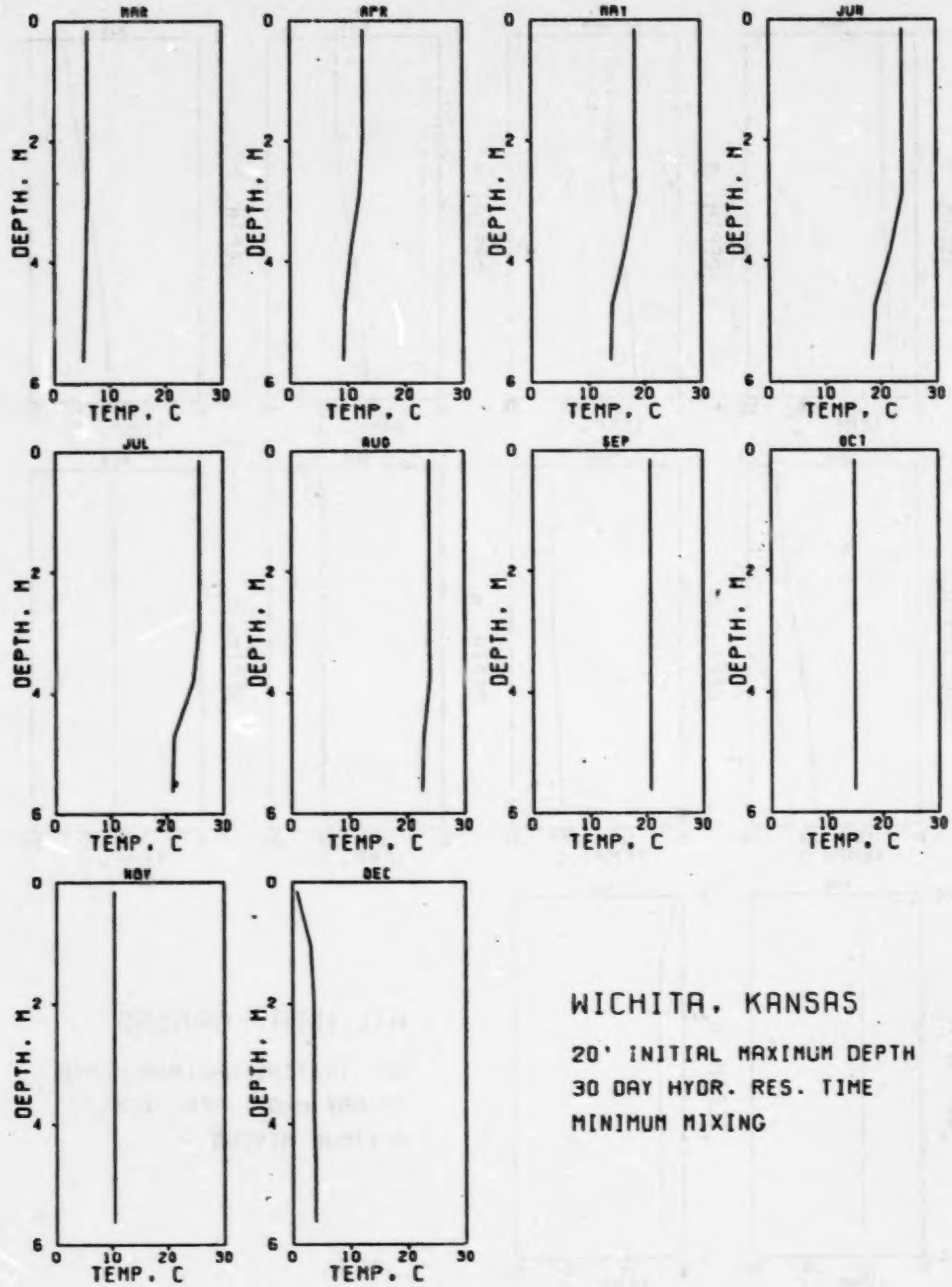
WICHITA, KANSAS

20' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



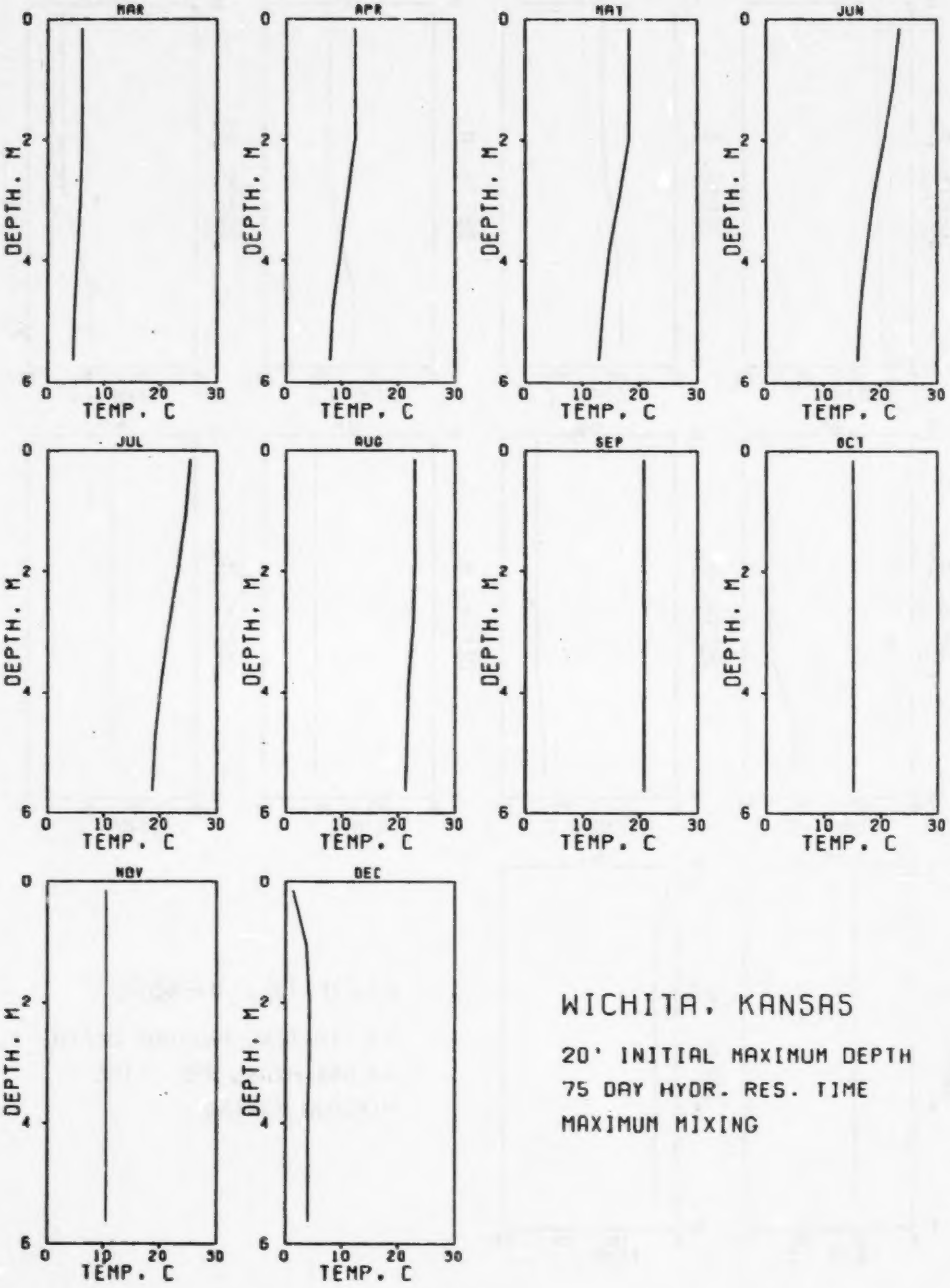
WICHITA, KANSAS

20' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



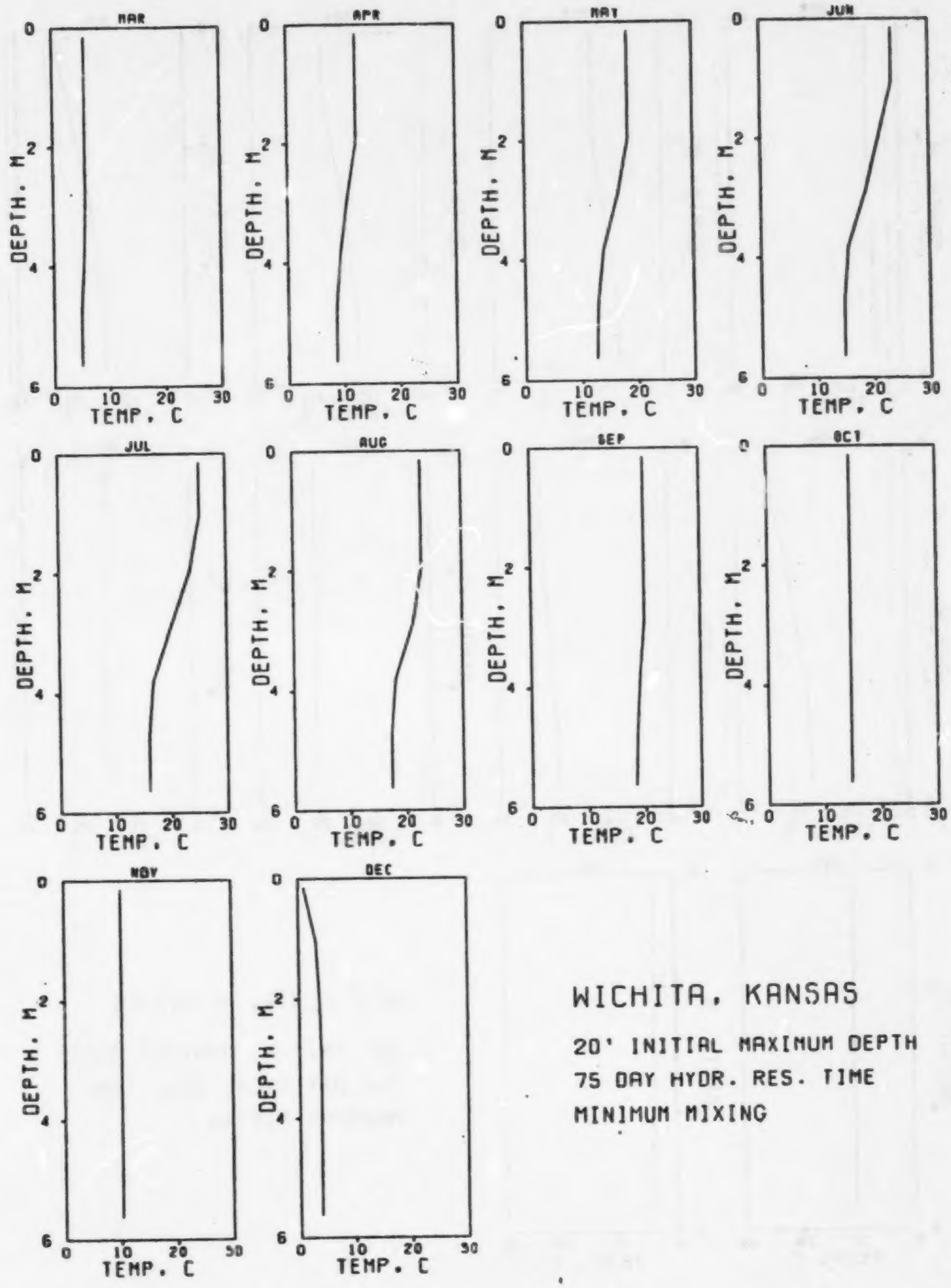
WICHITA, KANSAS

20' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



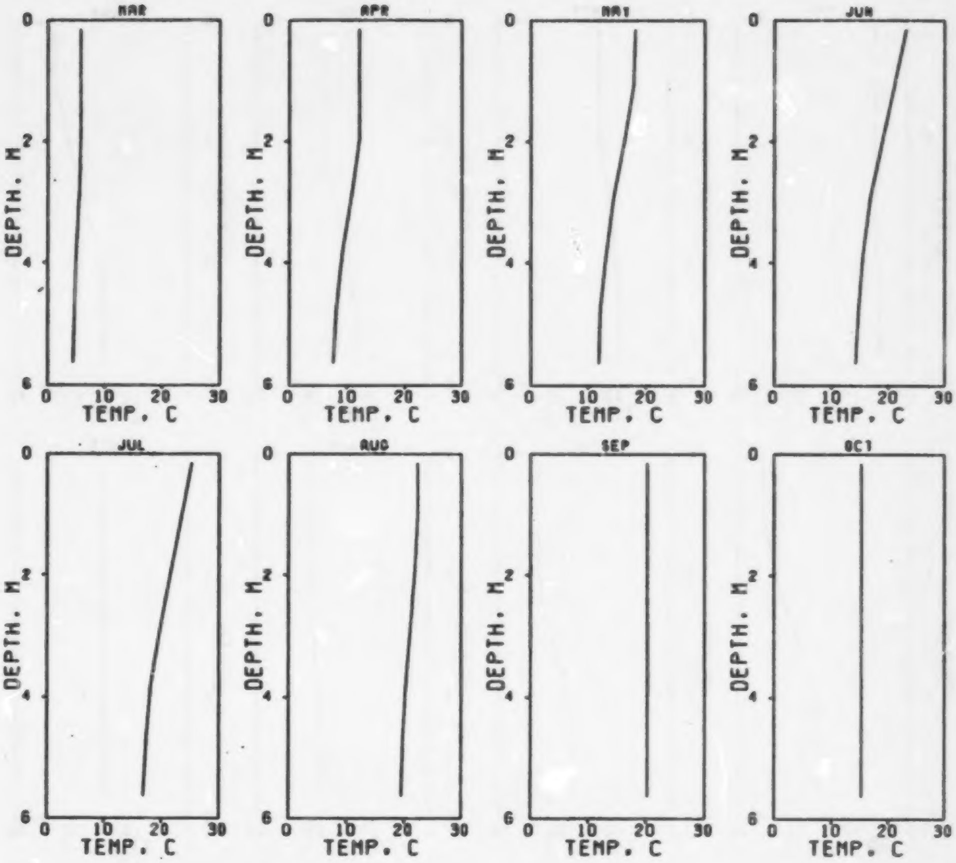
WICHITA, KANSAS

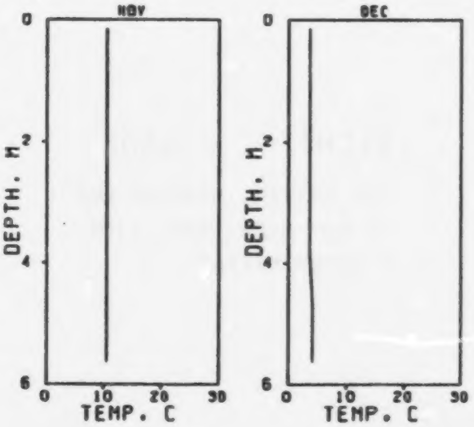
20' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



WICHITA, KANSAS

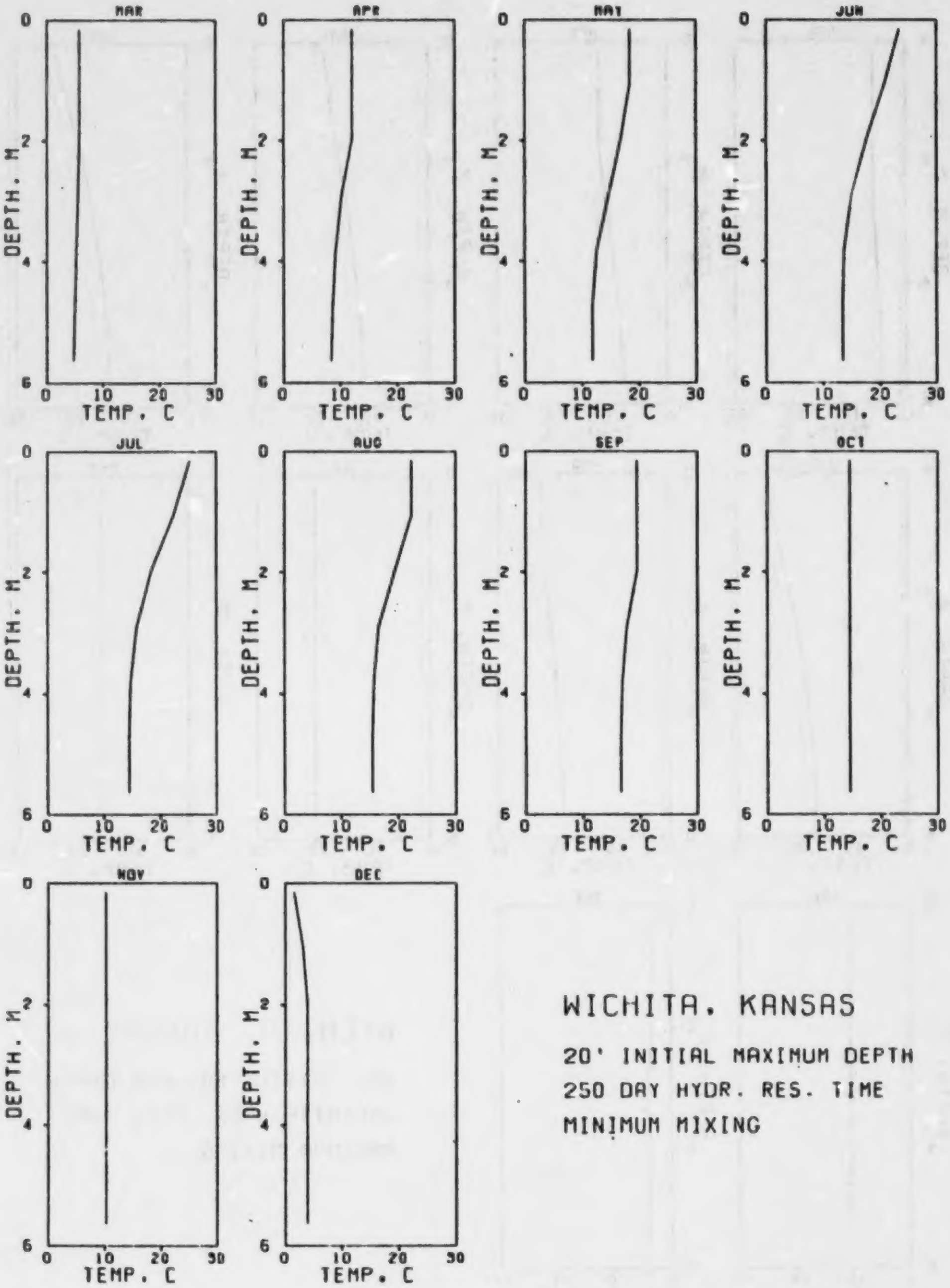
20' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING





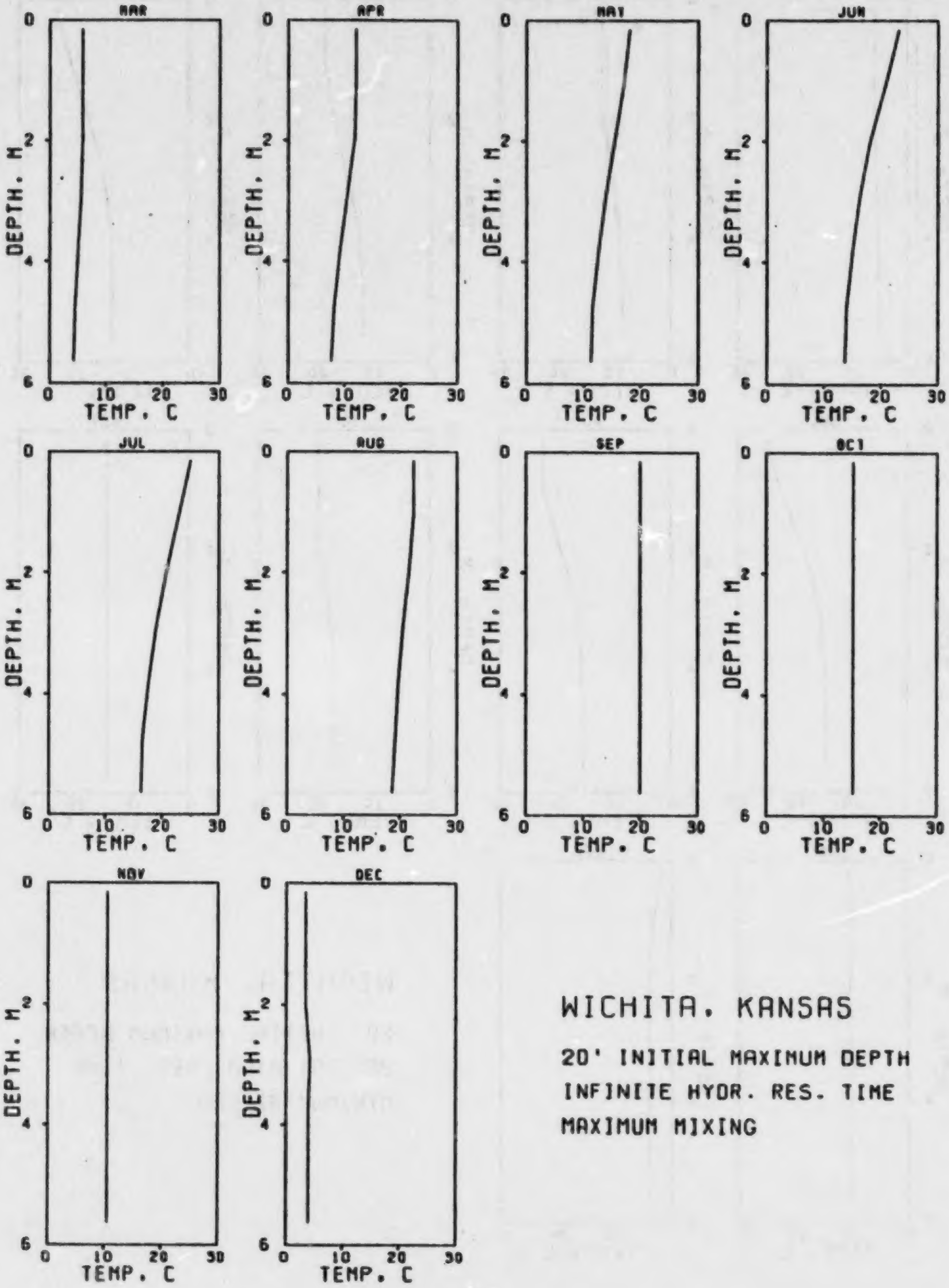
WICHITA, KANSAS

20' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



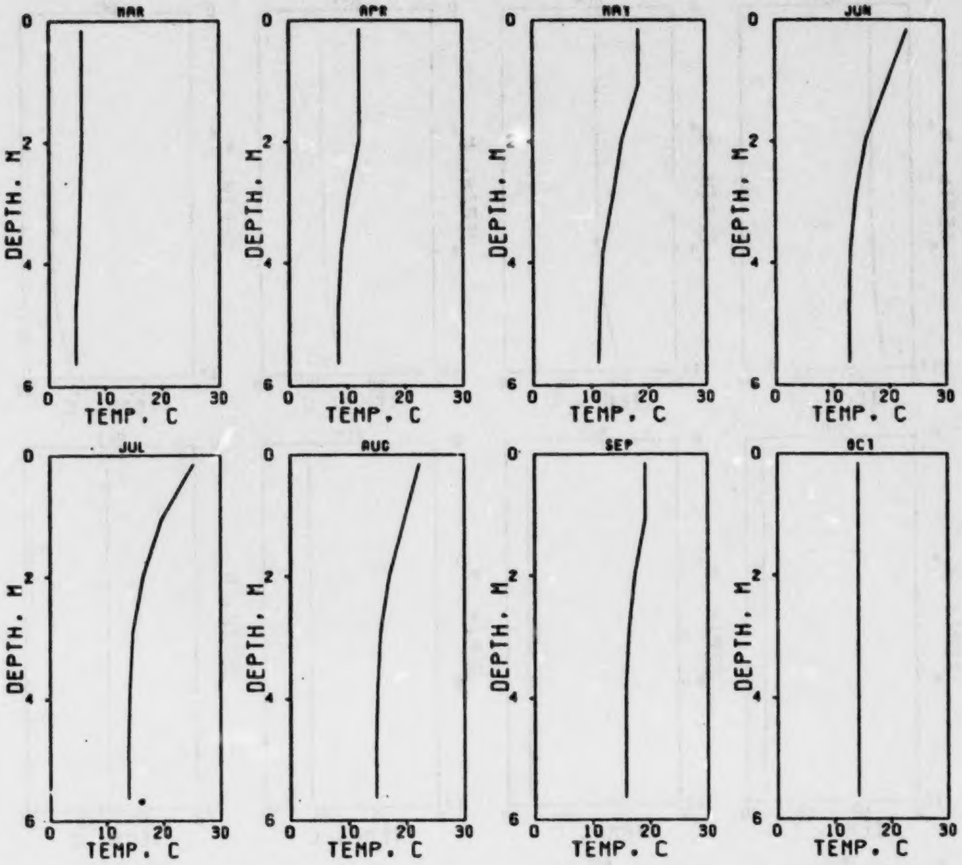
WICHITA, KANSAS

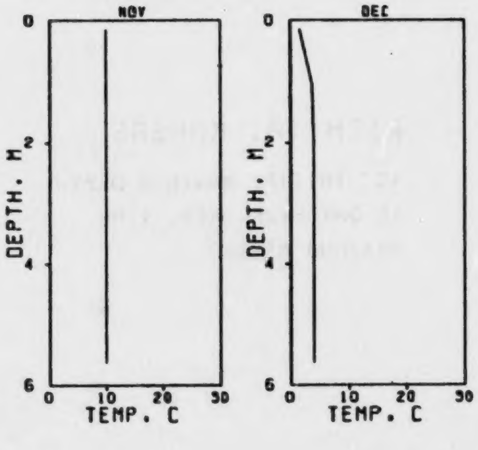
20' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



WICHITA, KANSAS

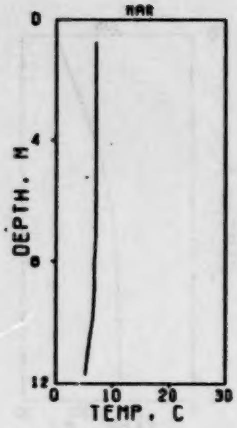
20' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING

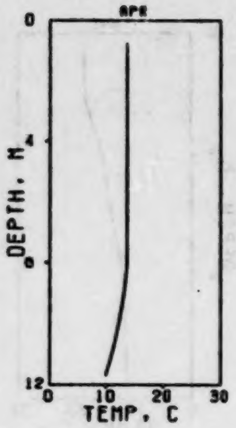


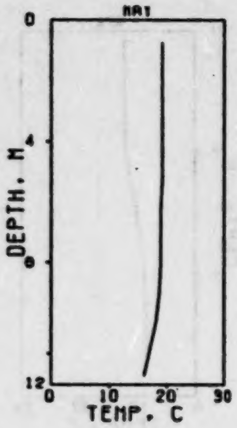


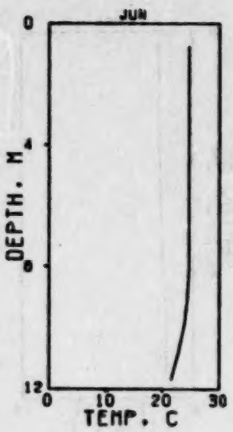
WICHITA. KANSAS

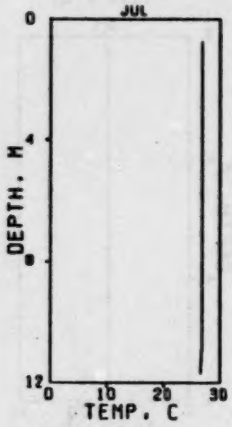
20' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING

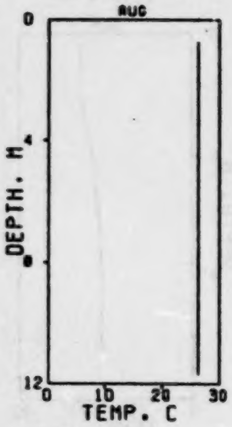


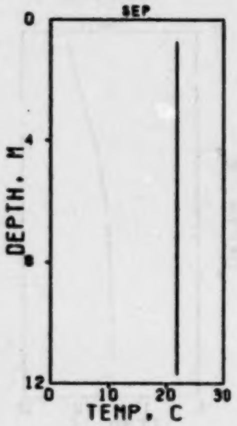


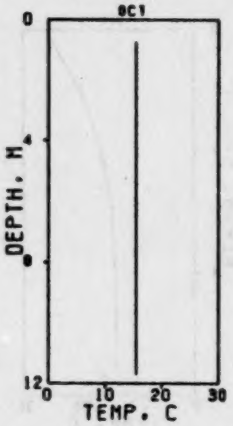


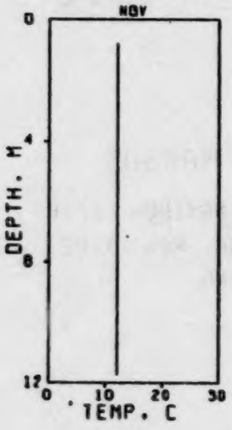


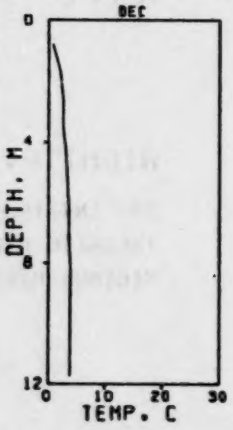






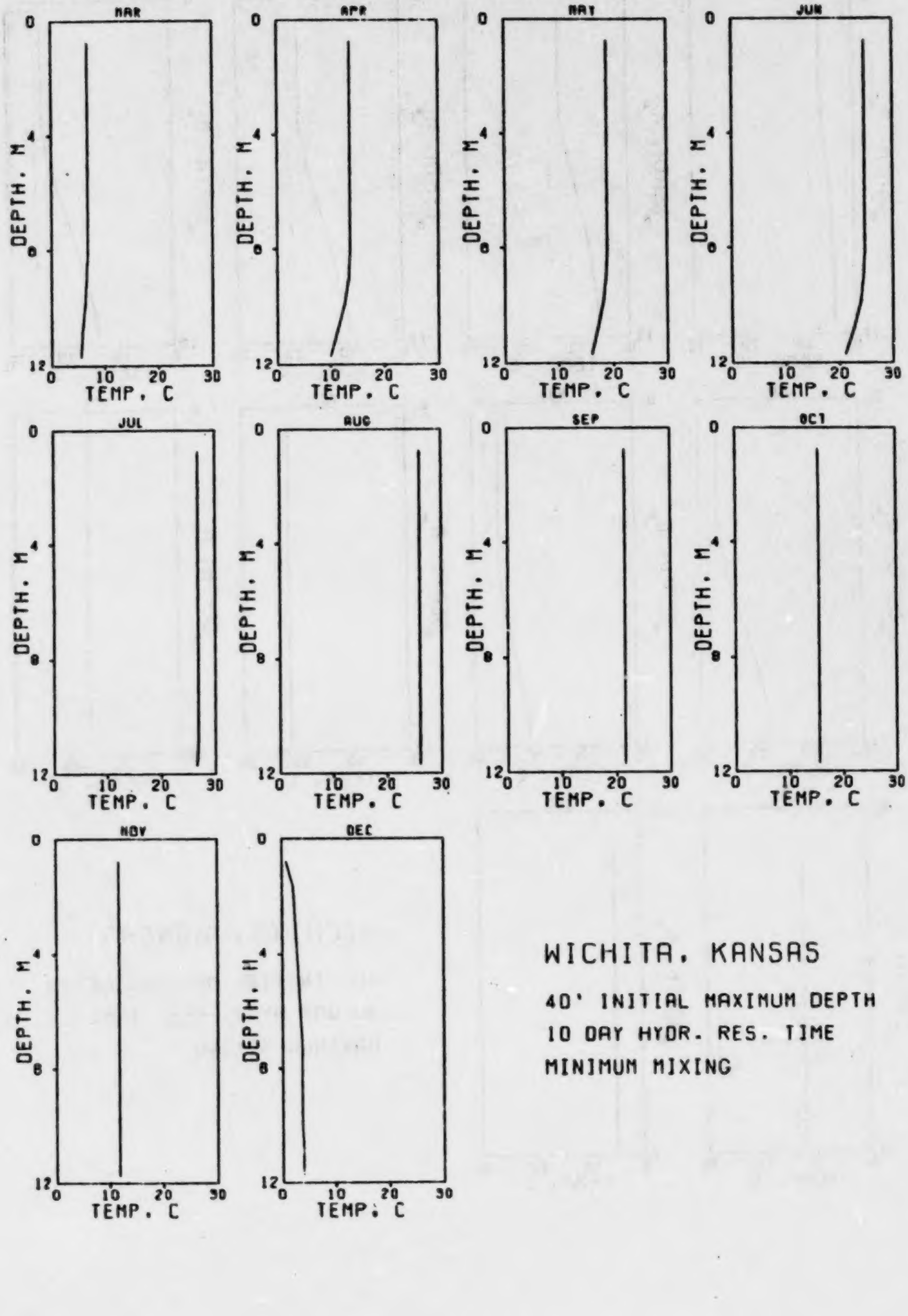






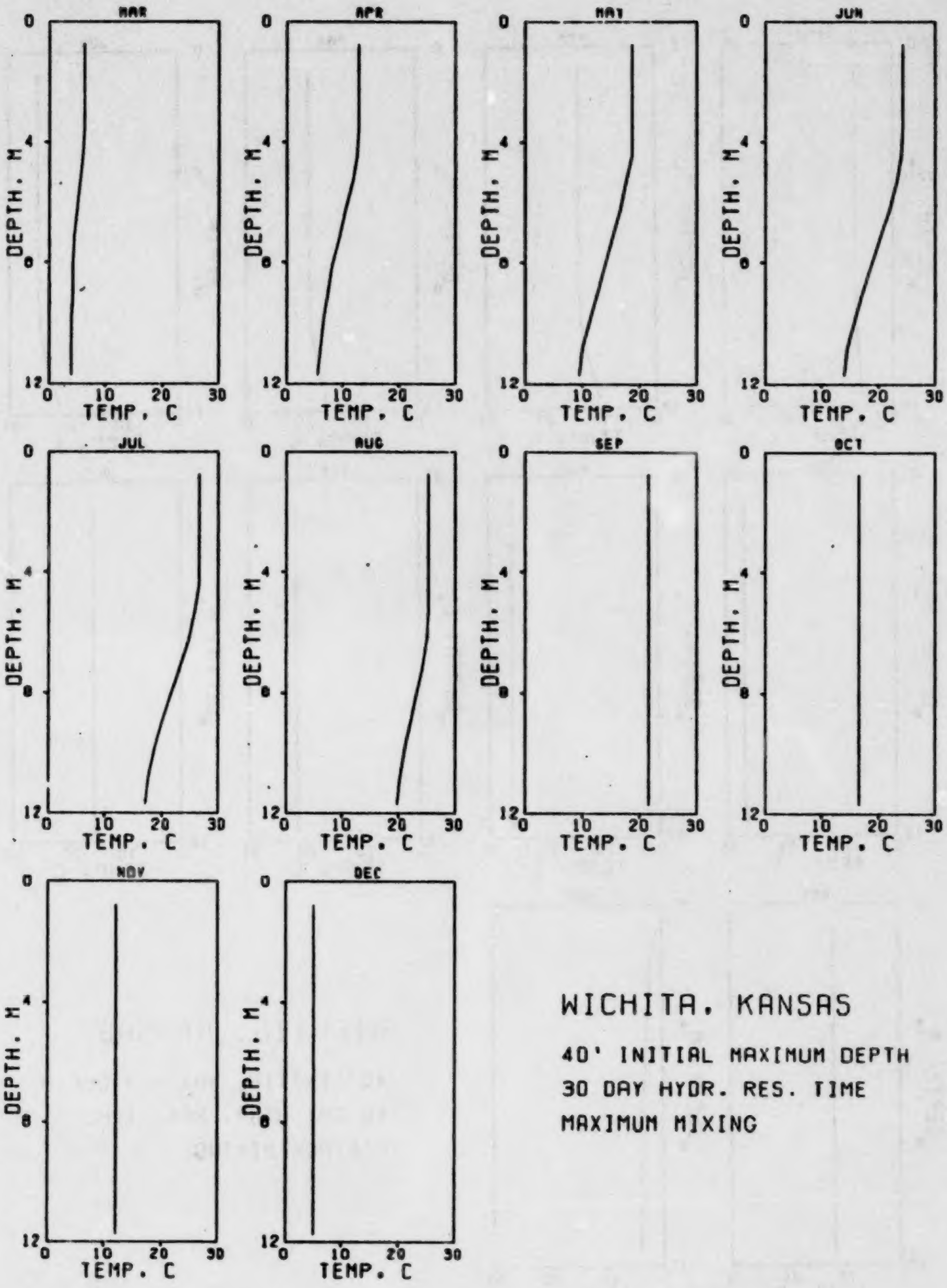
WICHITA, KANSAS

40' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



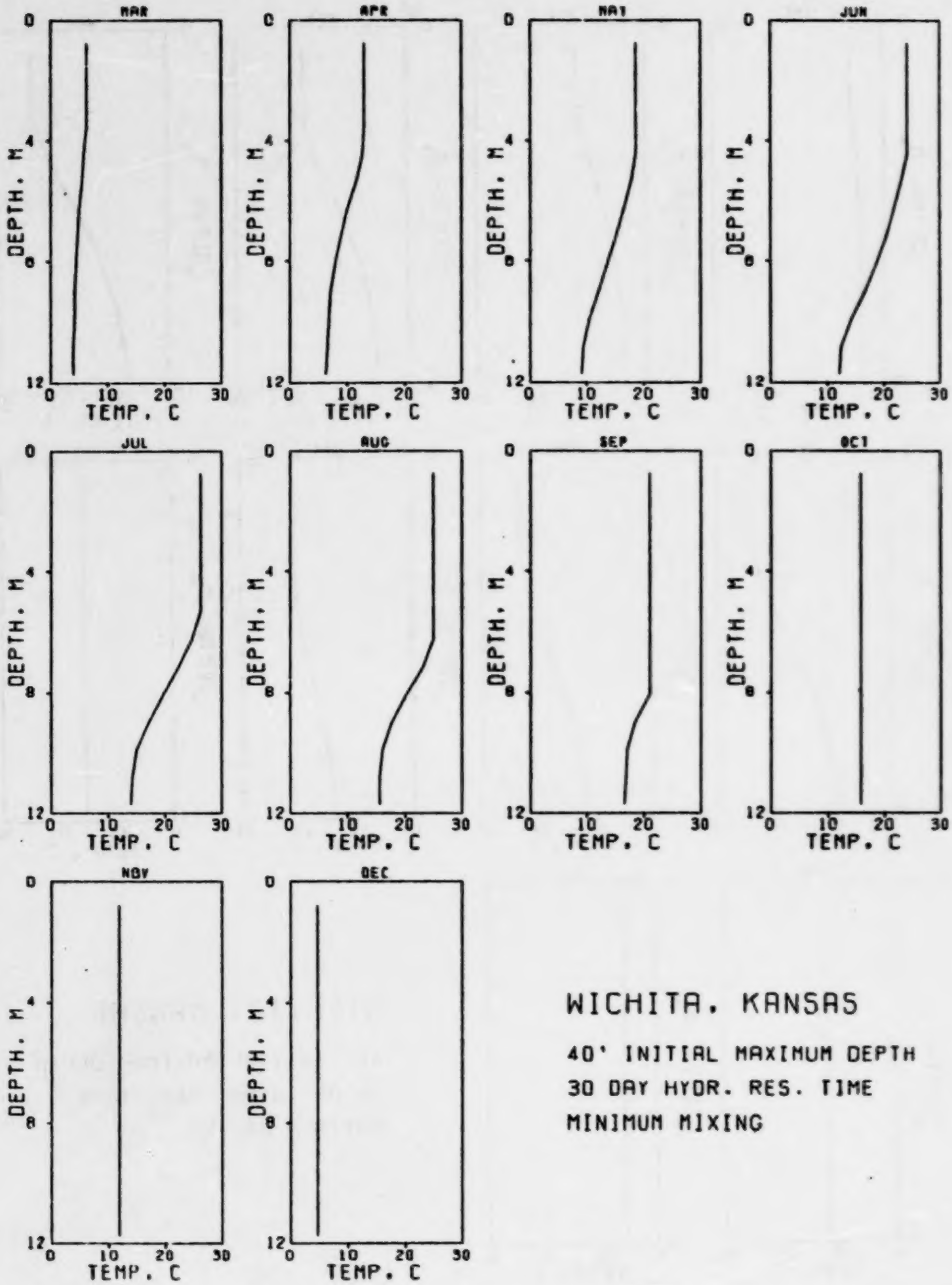
WICHITA, KANSAS

40' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



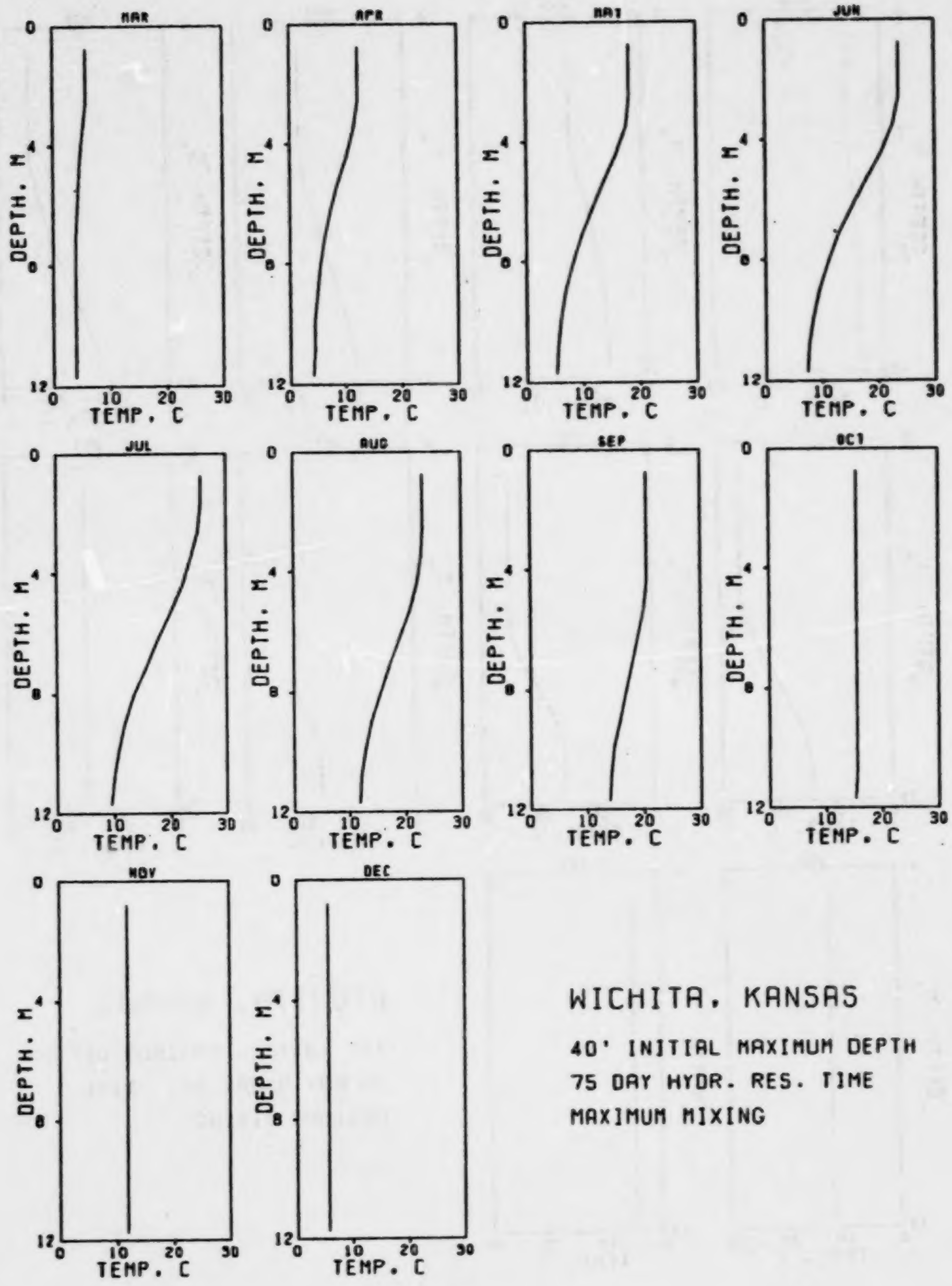
WICHITA, KANSAS

40' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



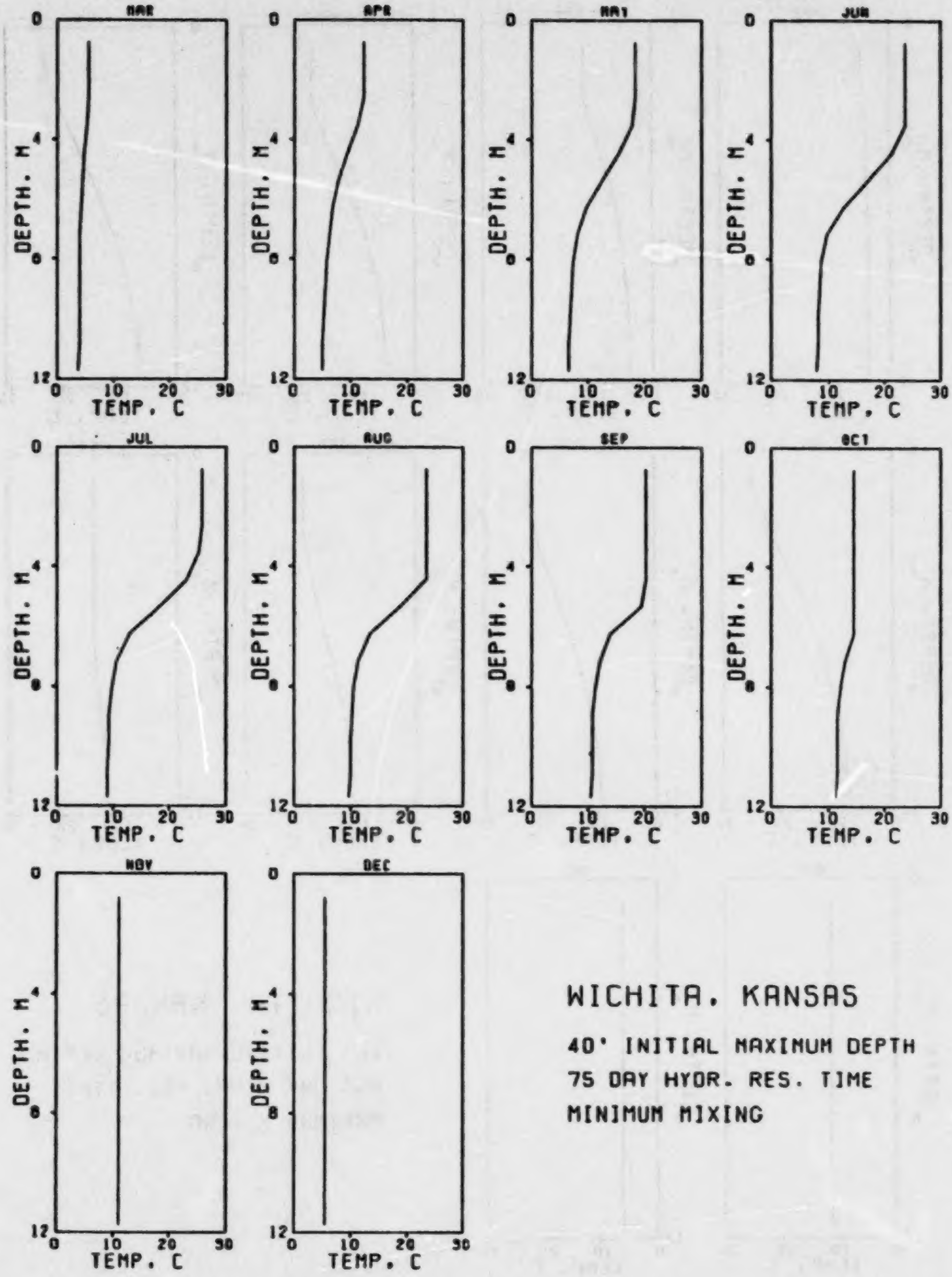
WICHITA, KANSAS

40' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



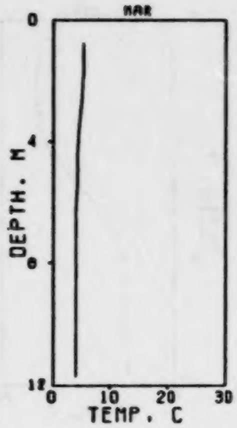
WICHITA, KANSAS

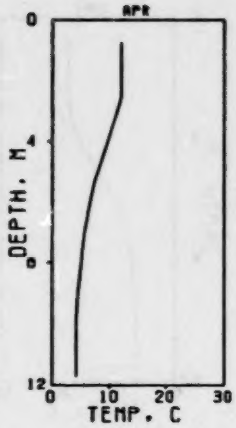
40' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING

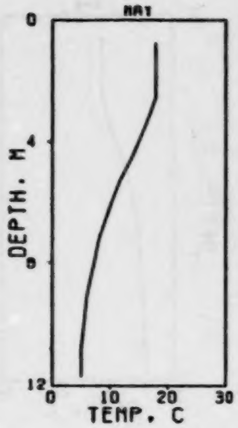


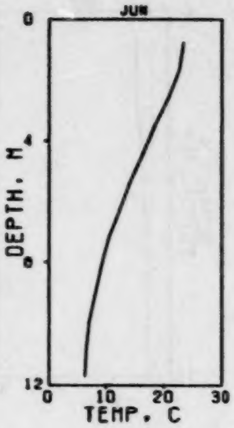
WICHITA, KANSAS

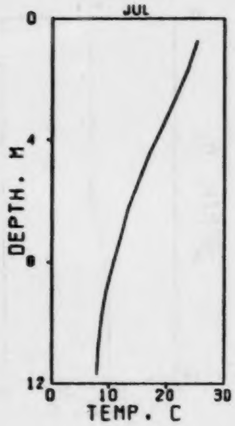
40' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING

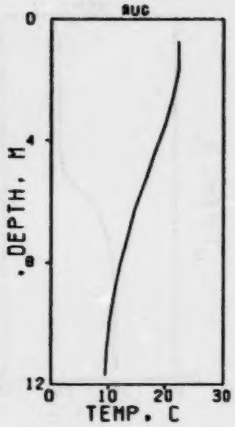


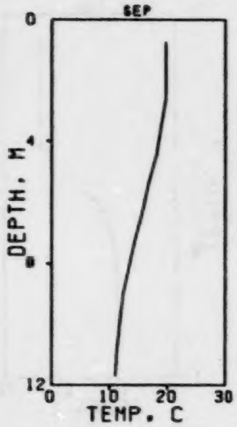


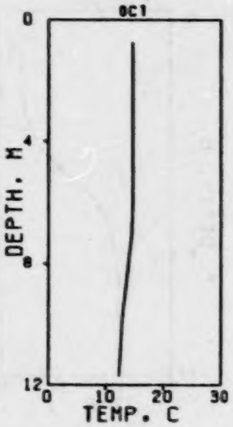


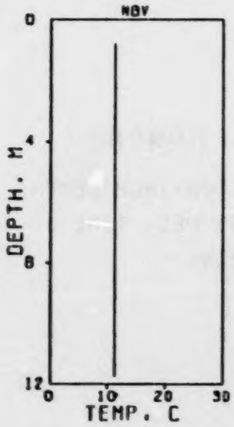


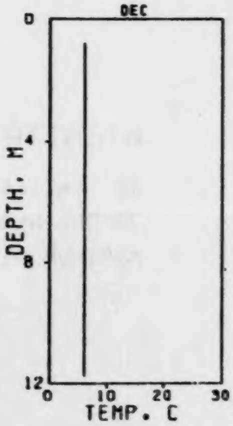






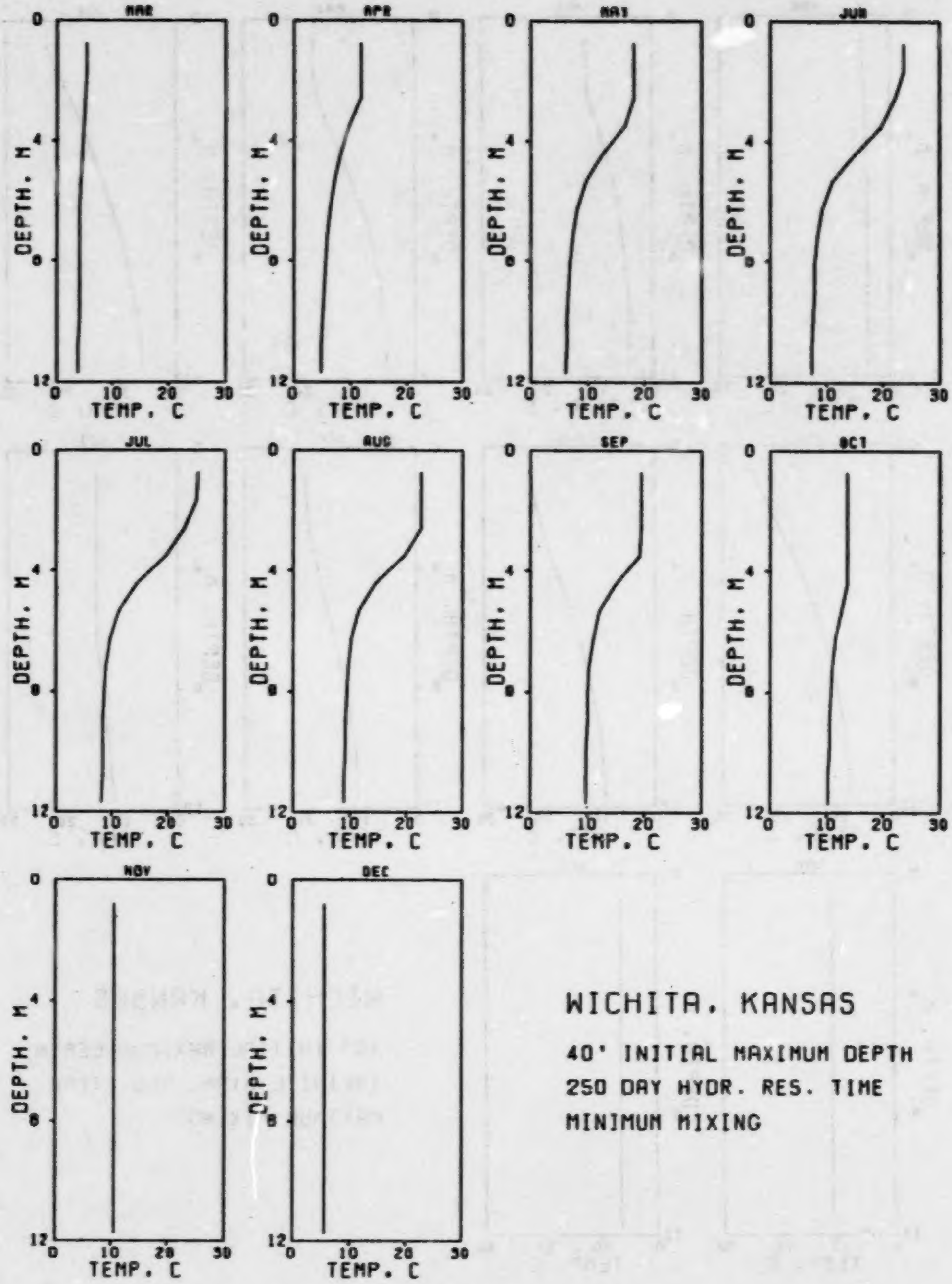






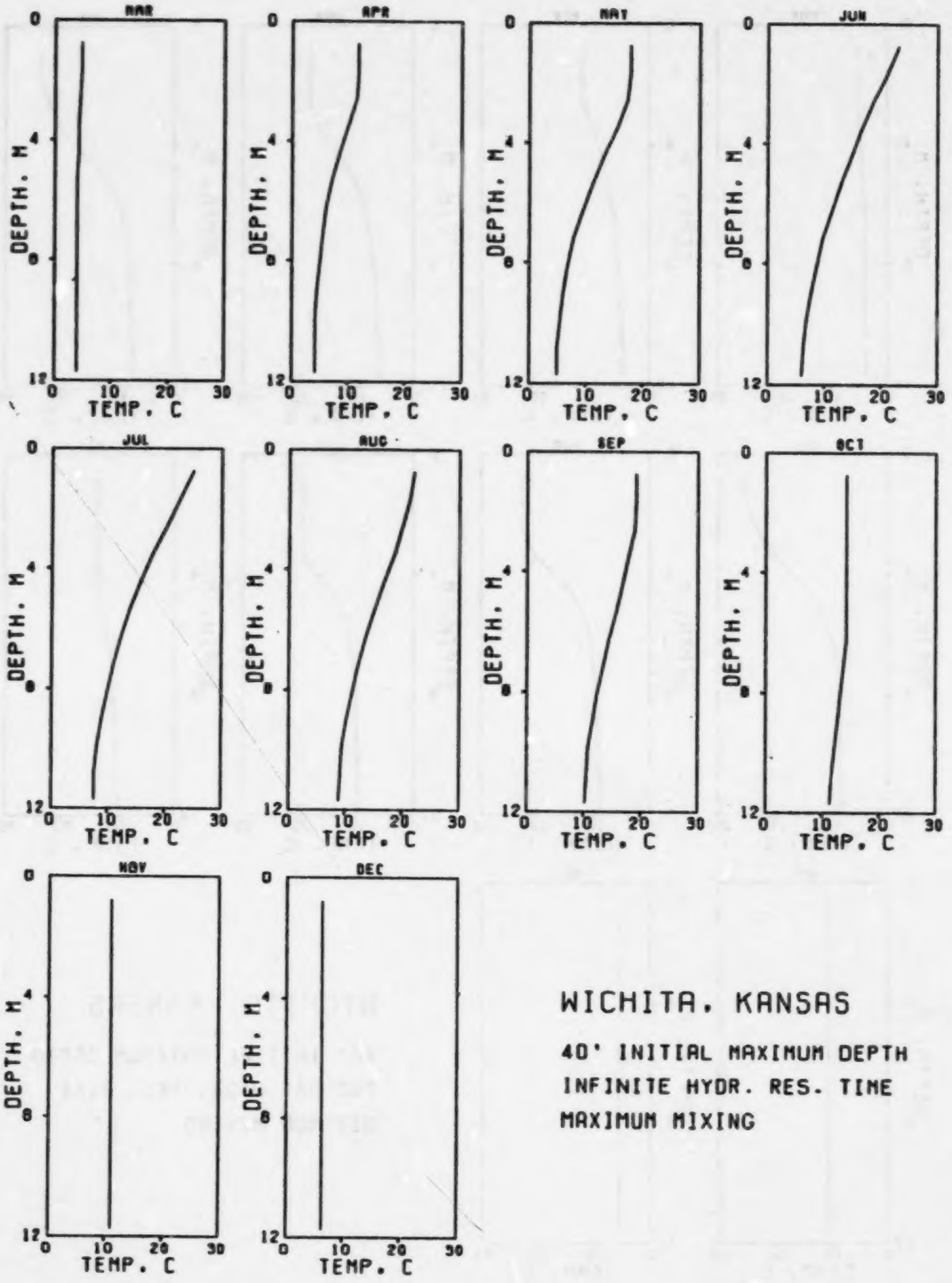
WICHITA, KANSAS

40' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



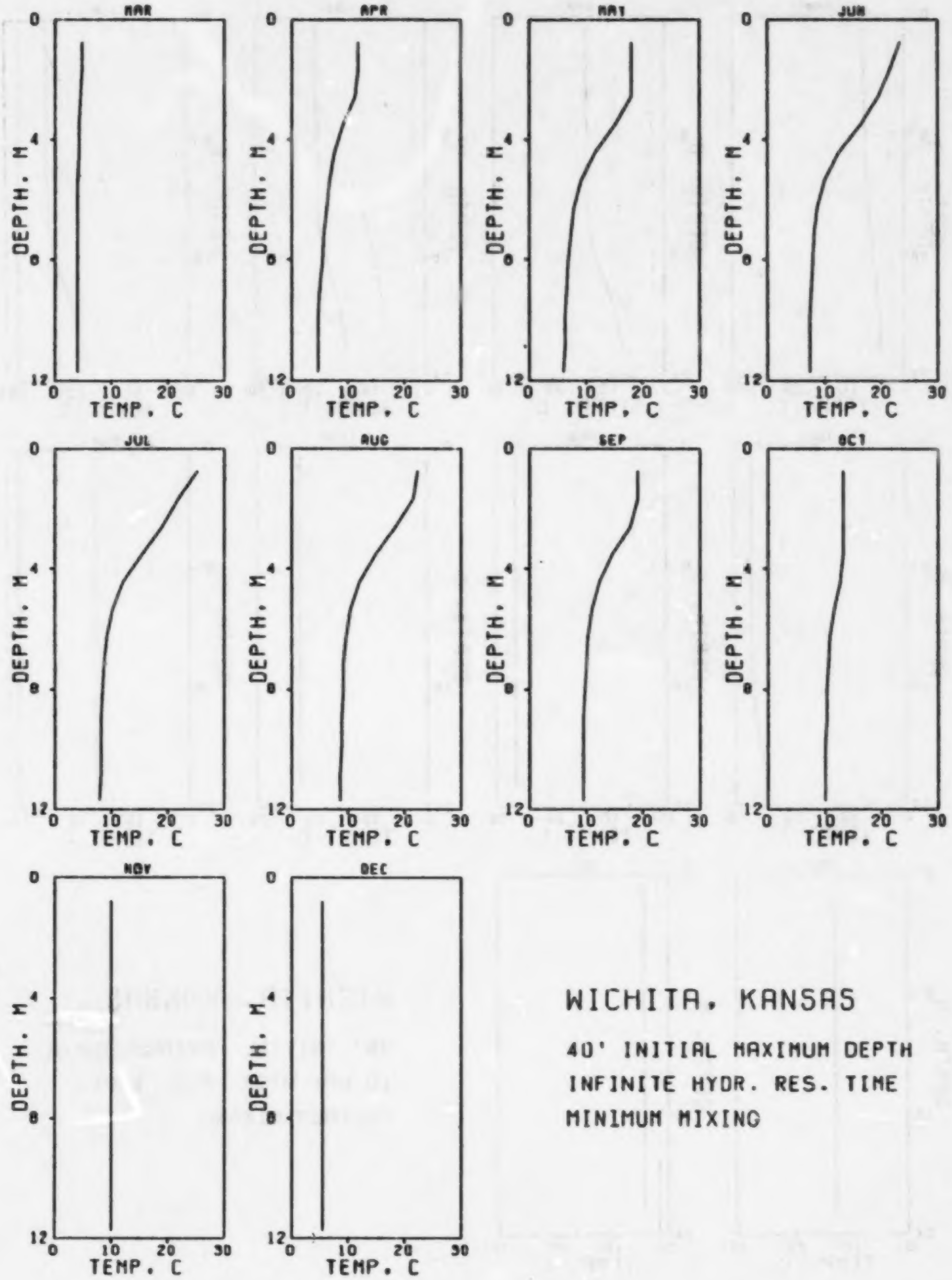
WICHITA, KANSAS

40' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



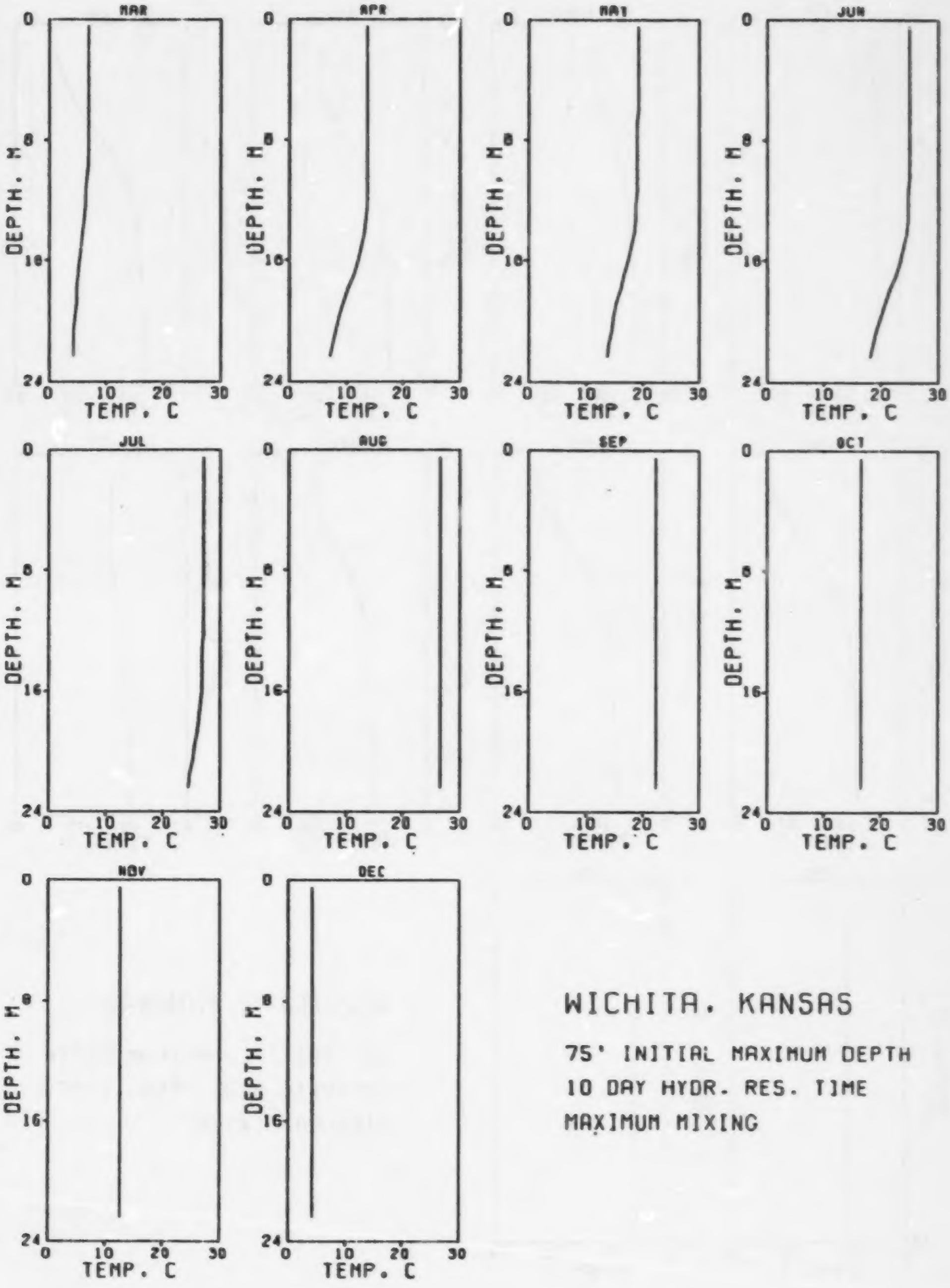
WICHITA, KANSAS

40' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



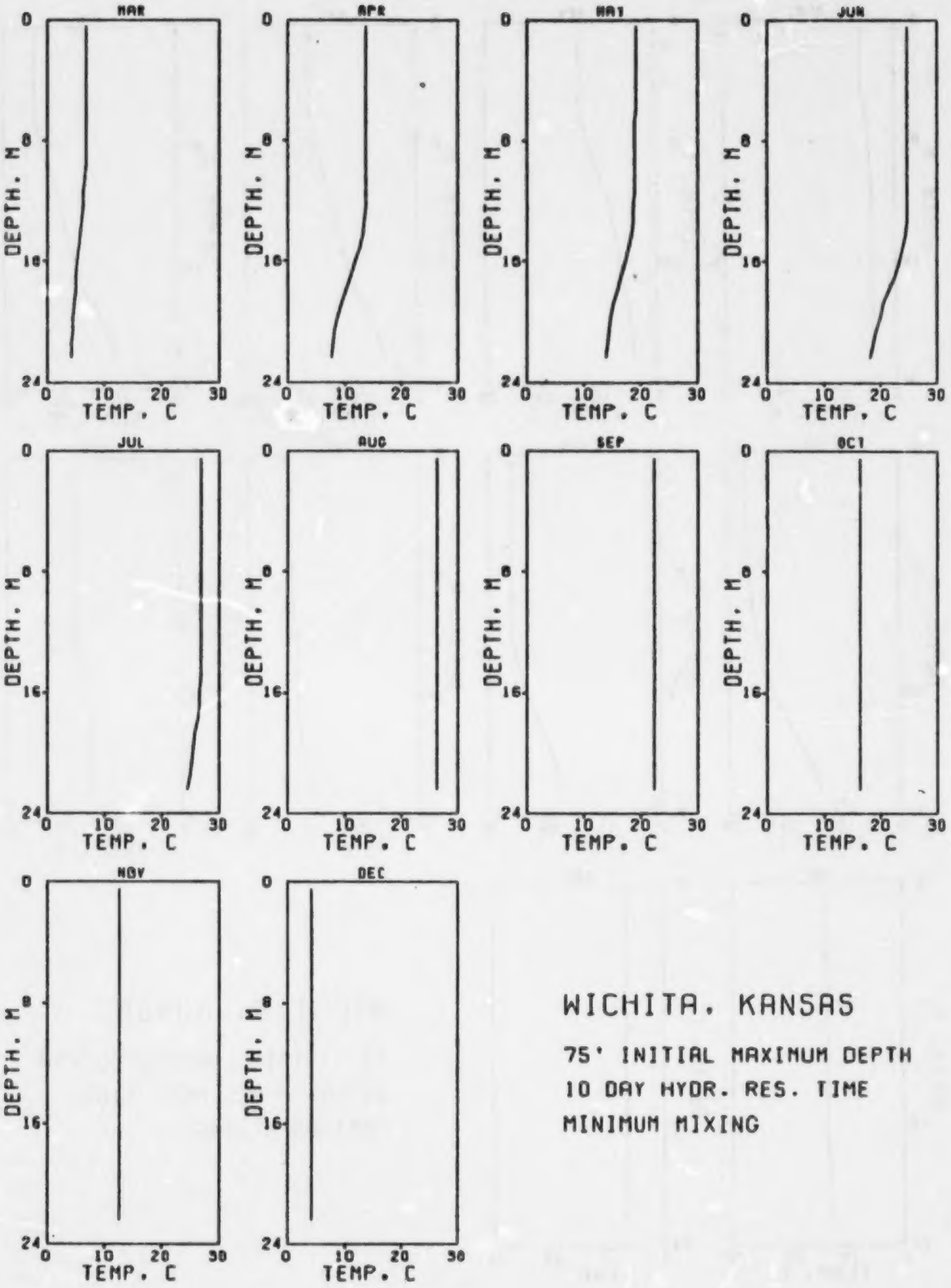
WICHITA, KANSAS

40' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



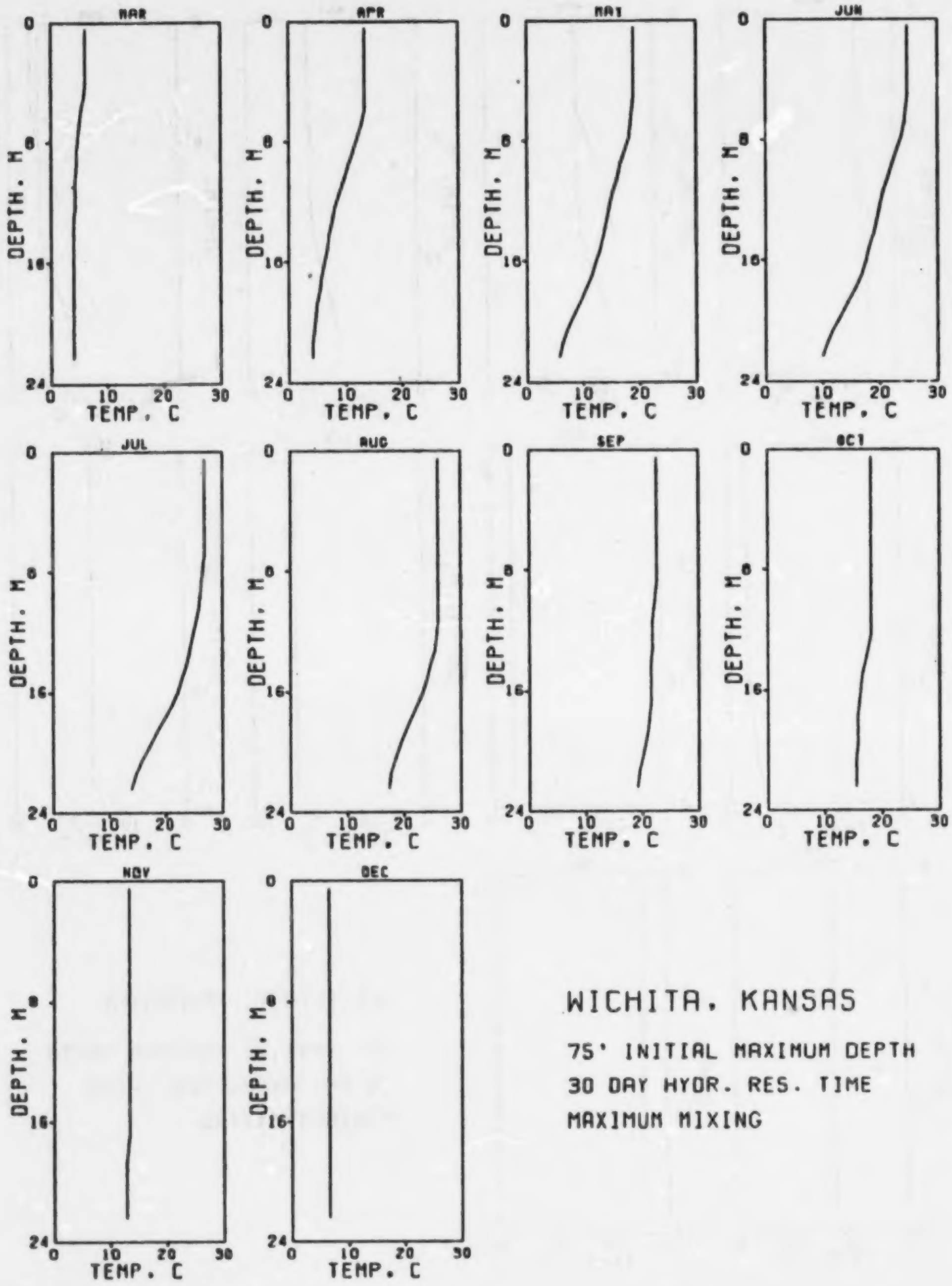
WICHITA, KANSAS

75' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



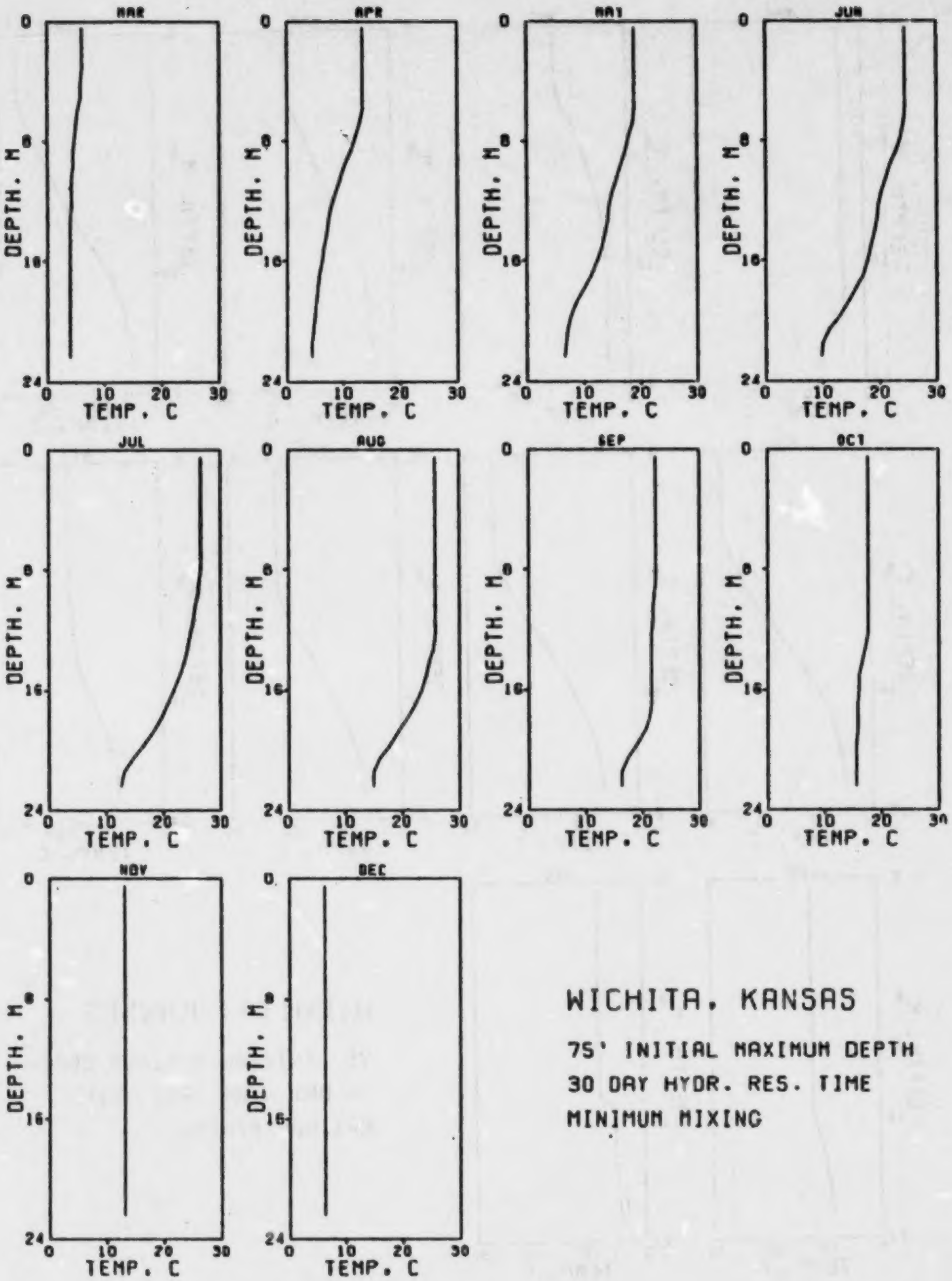
WICHITA, KANSAS

75' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



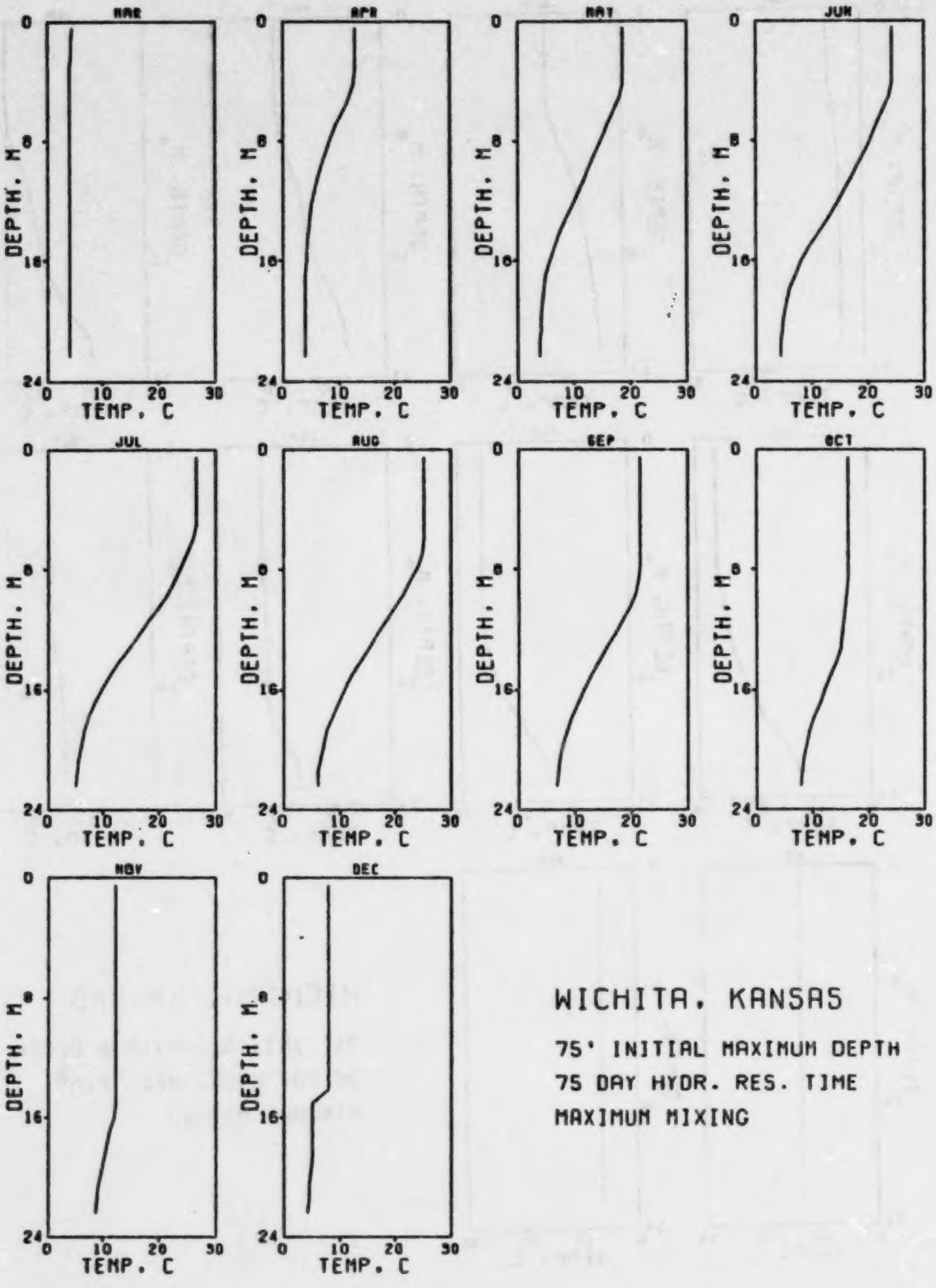
WICHITA, KANSAS

75' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



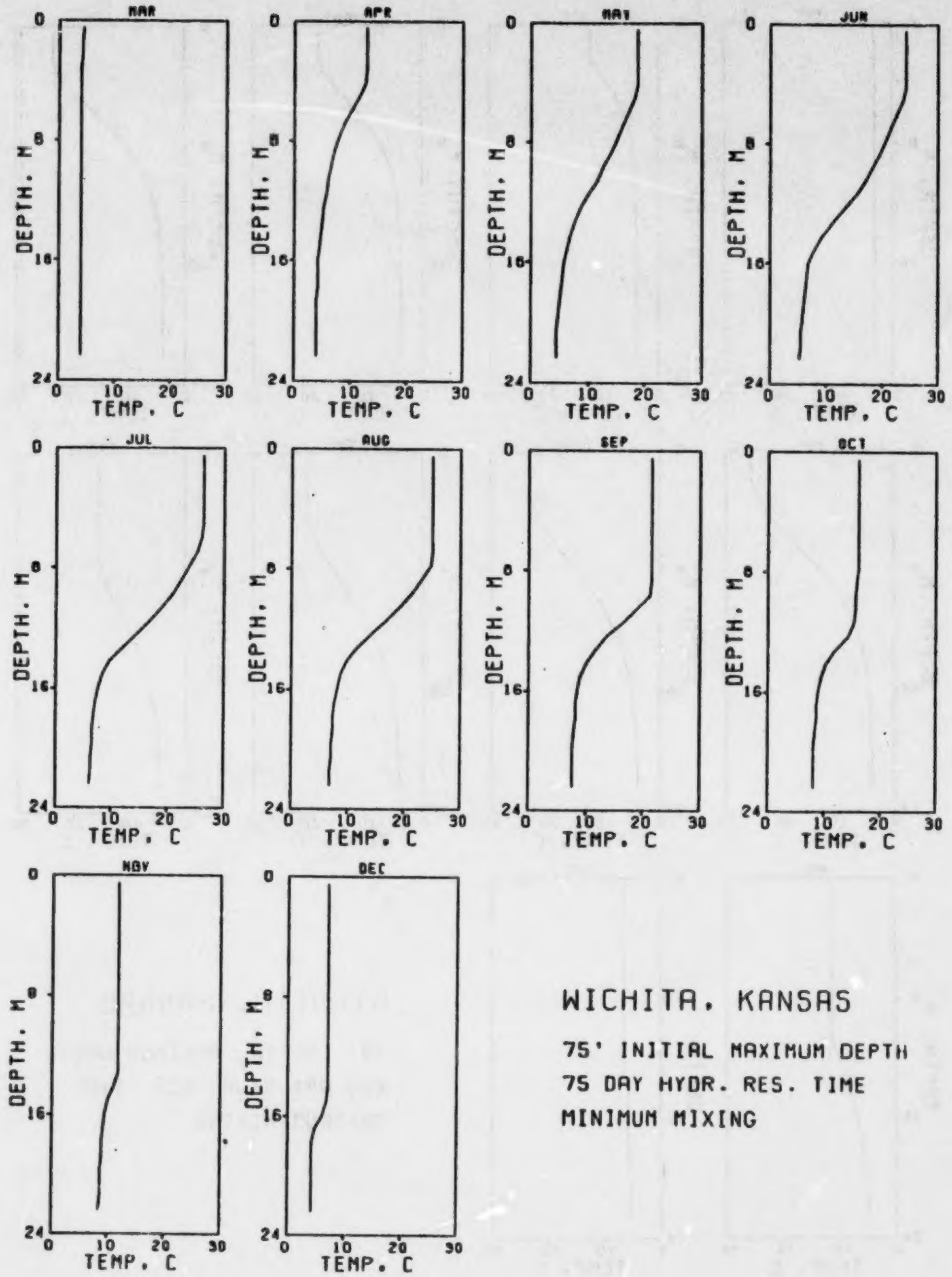
WICHITA, KANSAS

75' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



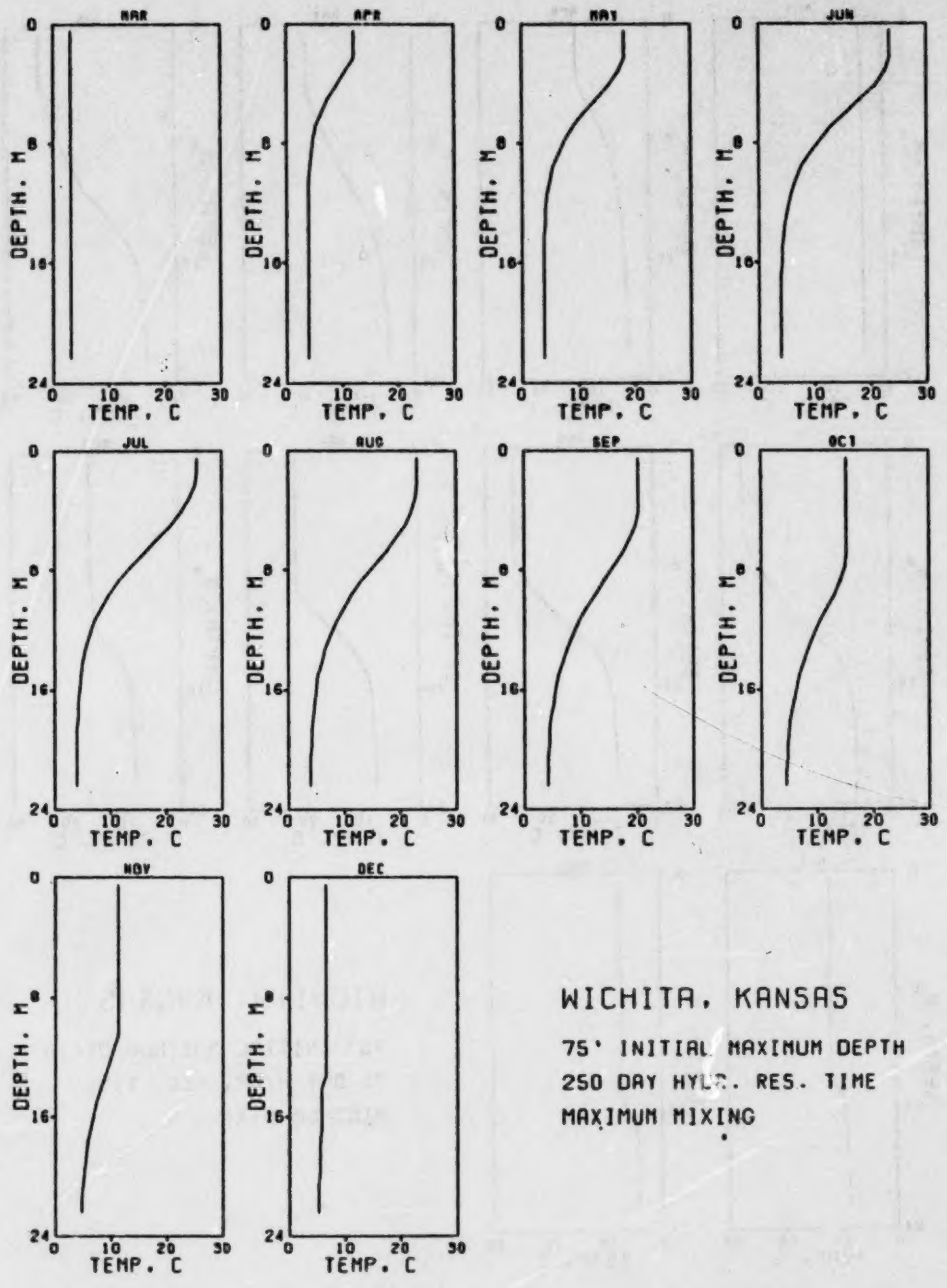
WICHITA, KANSAS

75' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



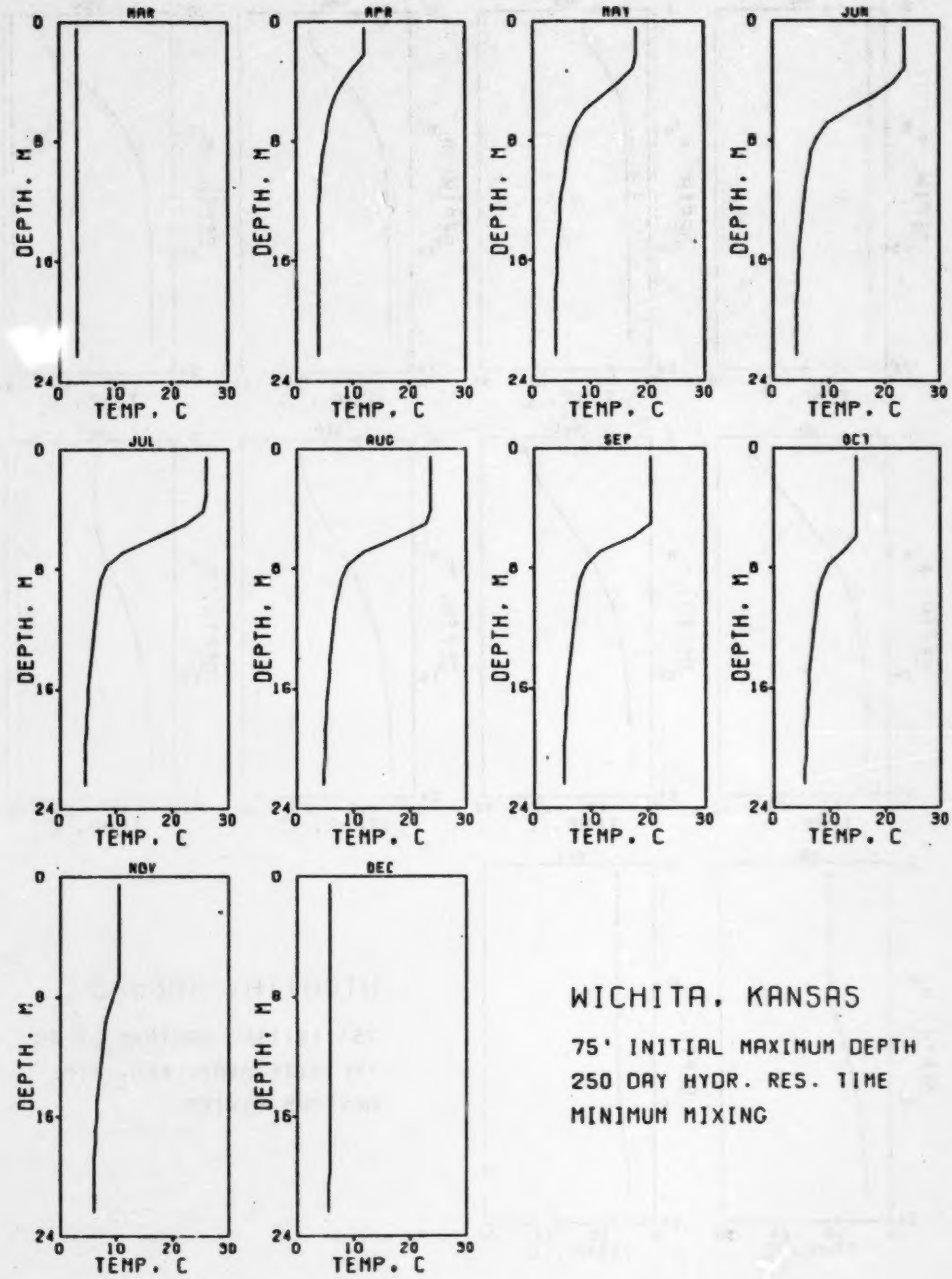
WICHITA, KANSAS

75' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



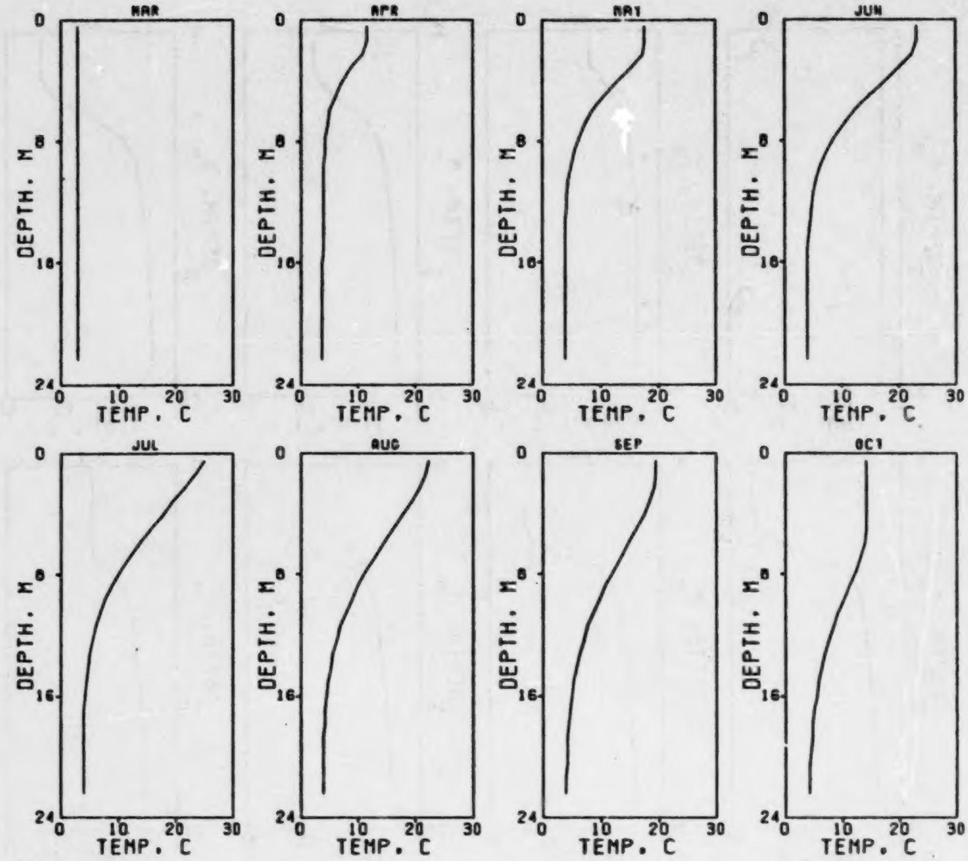
WICHITA, KANSAS

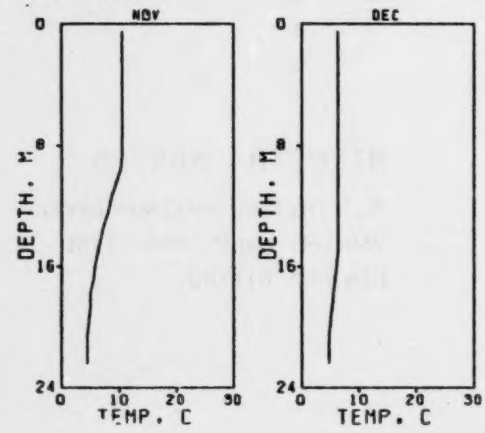
75' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



WICHITA, KANSAS

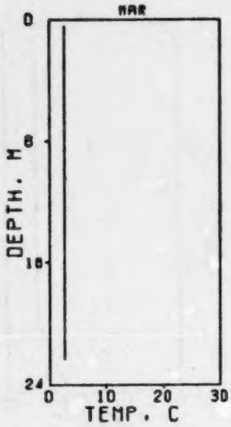
75' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING

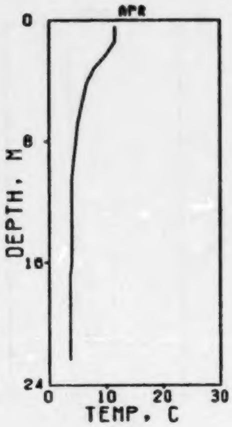


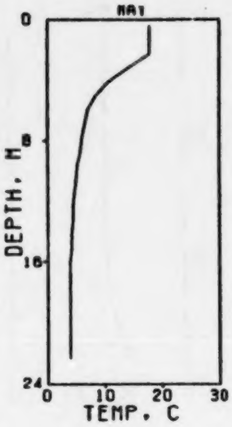


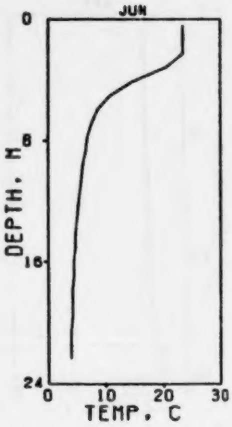
WICHITA, KANSAS

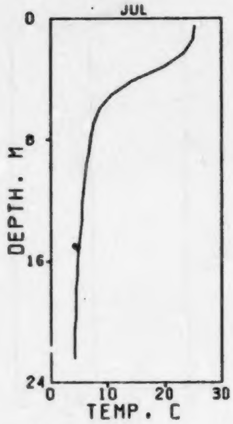
75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING

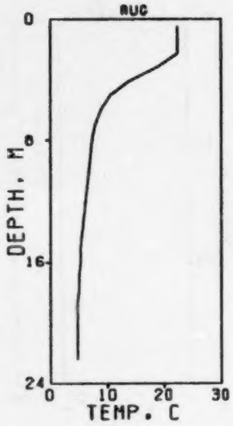


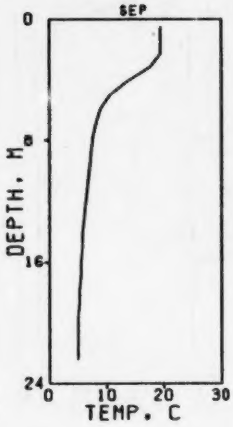


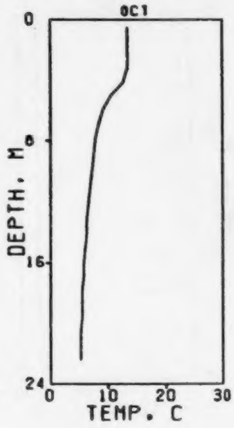


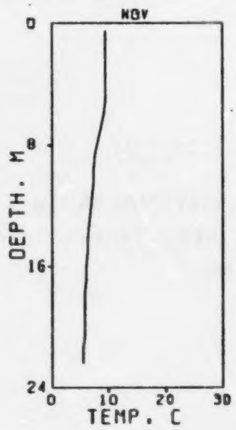


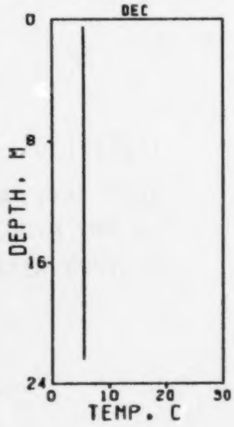






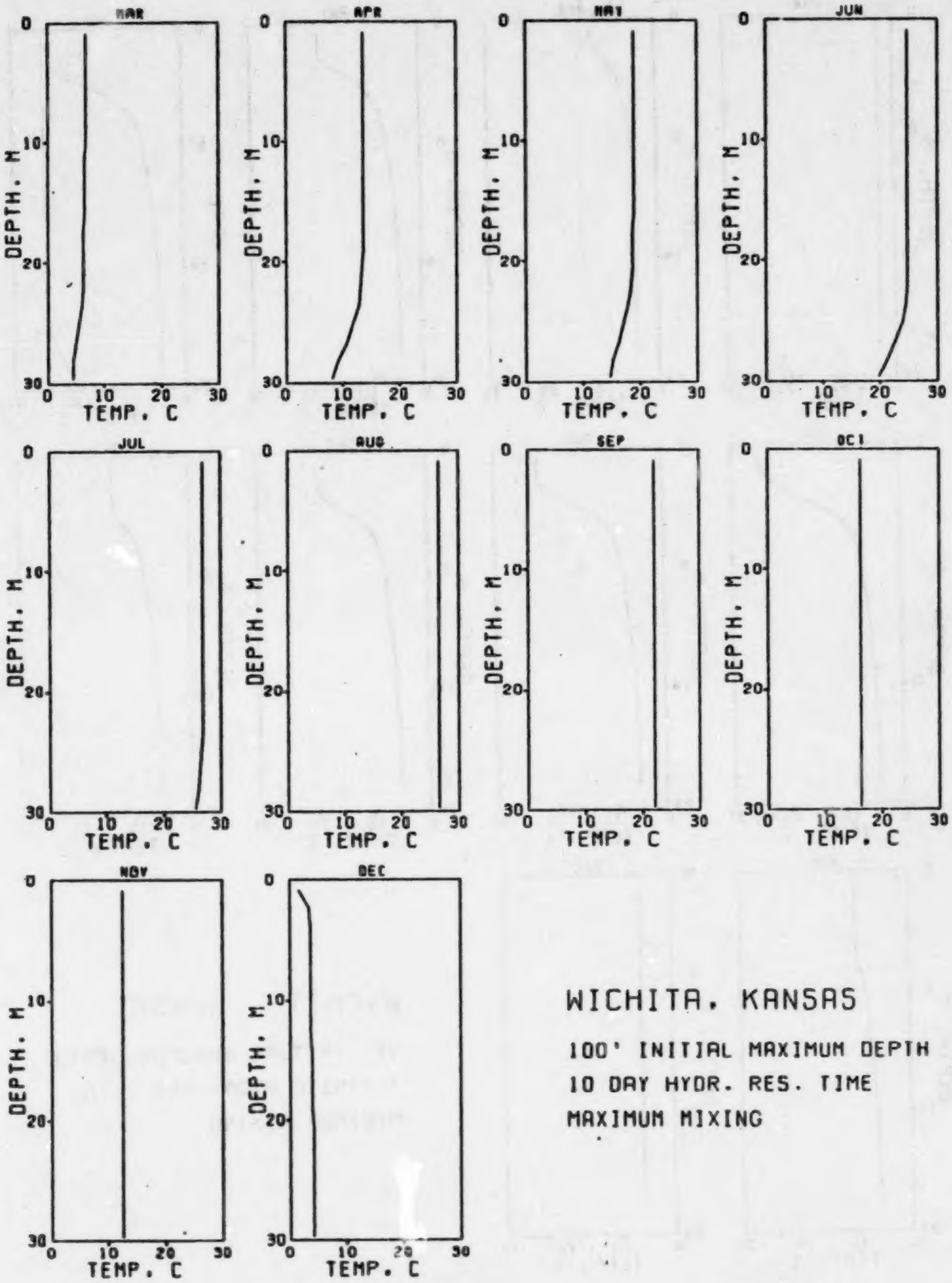






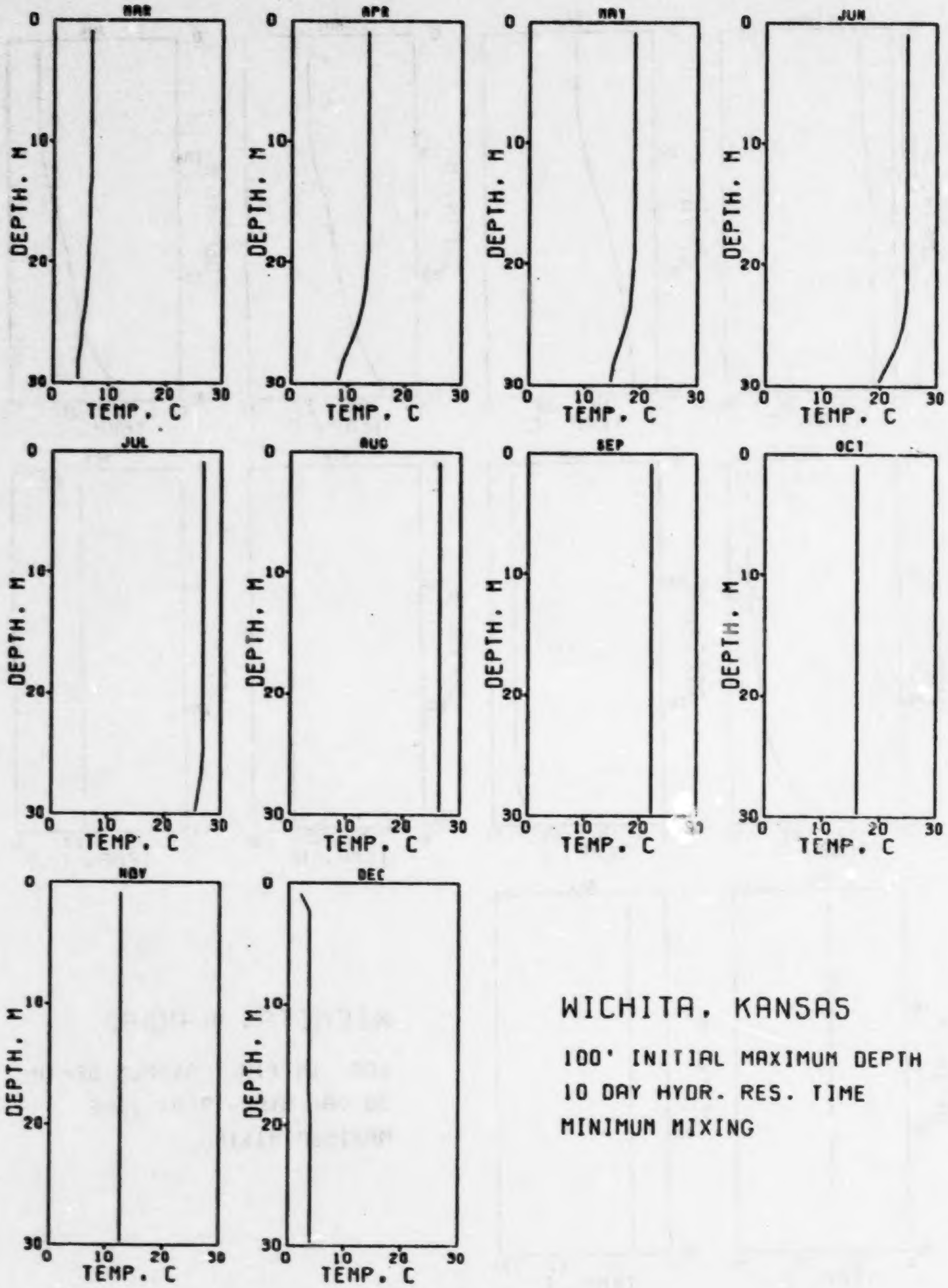
WICHITA, KANSAS

75' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



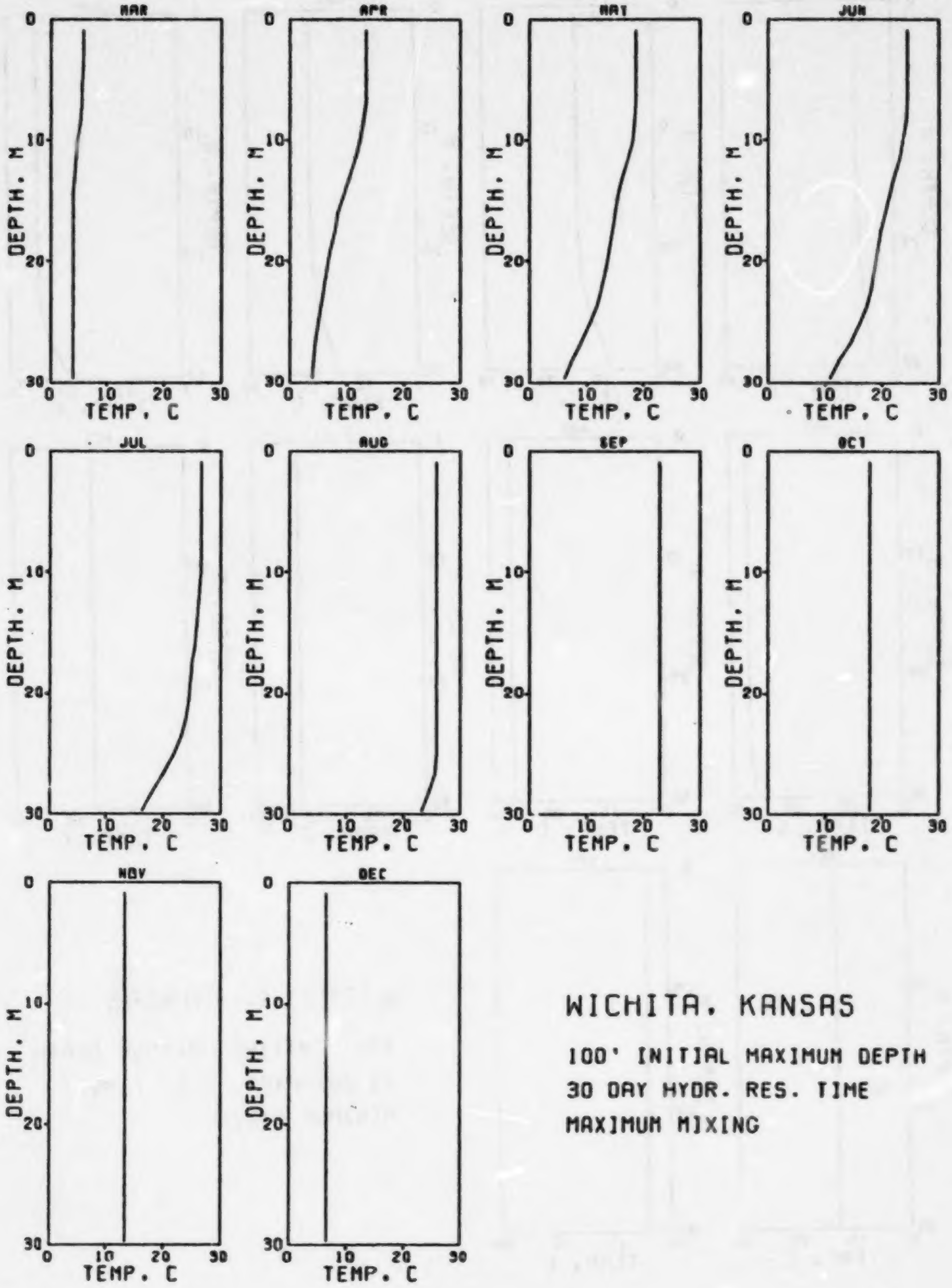
WICHITA, KANSAS

100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



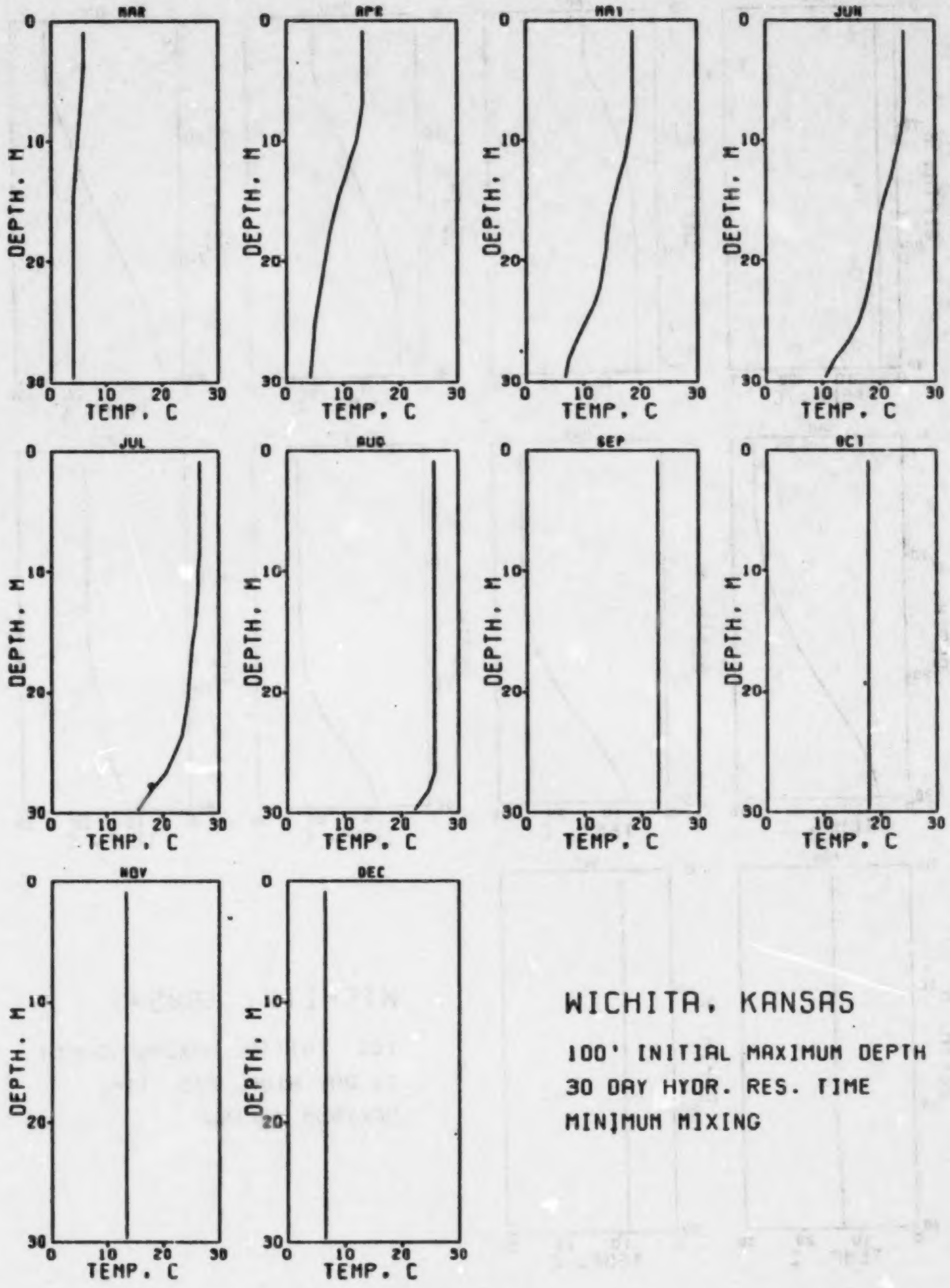
WICHITA, KANSAS

100' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



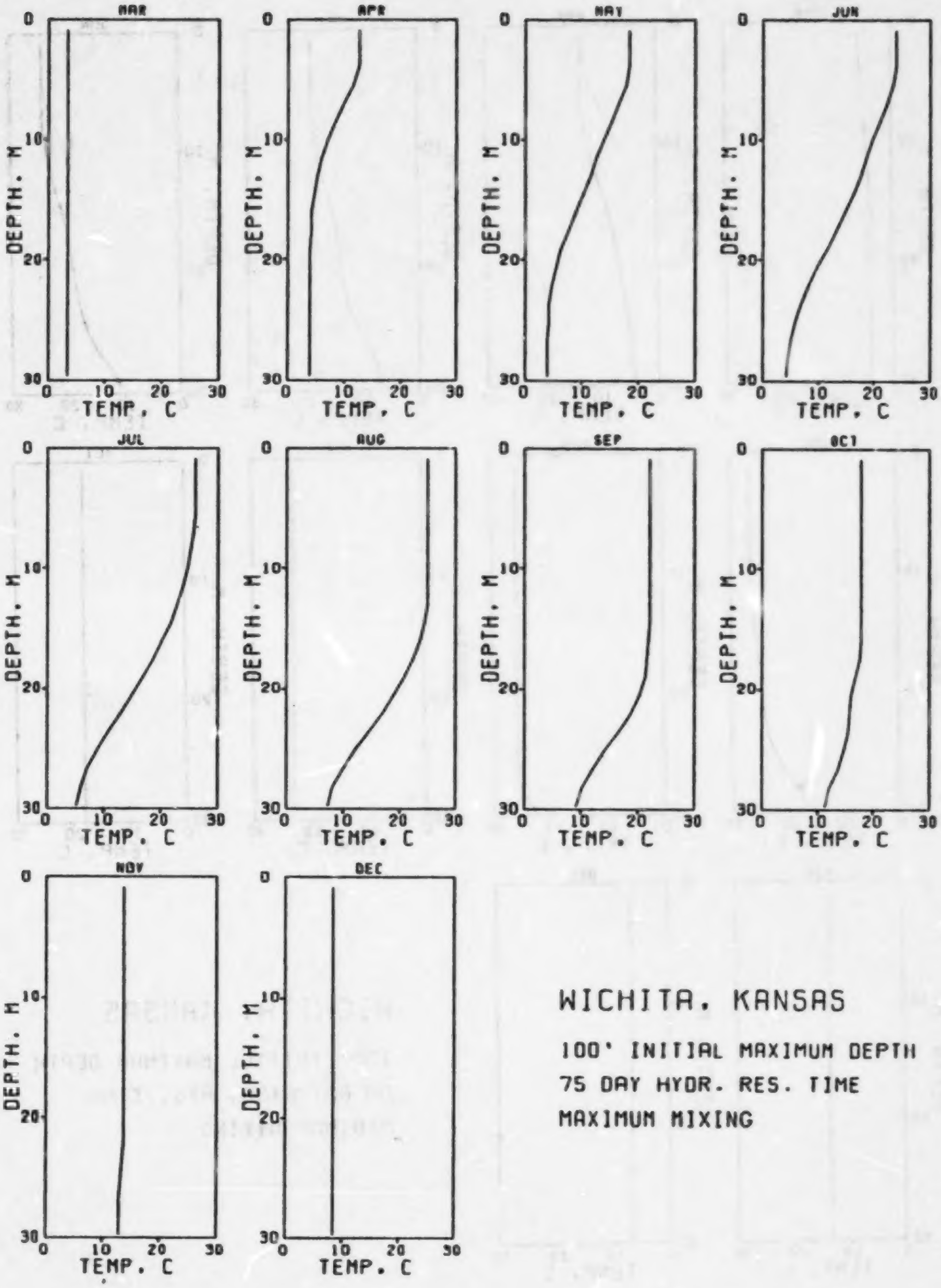
WICHITA, KANSAS

100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



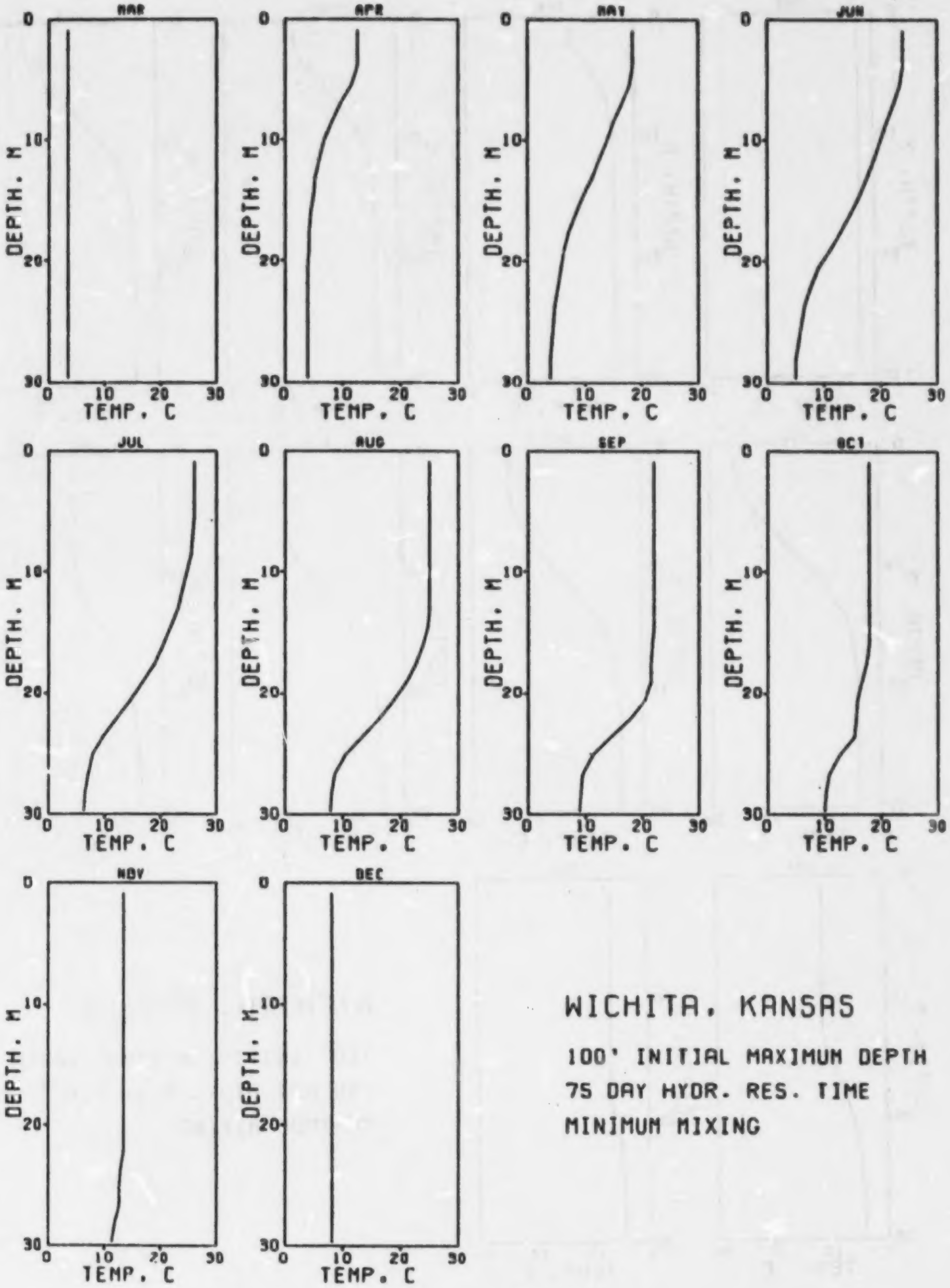
WICHITA, KANSAS

100' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



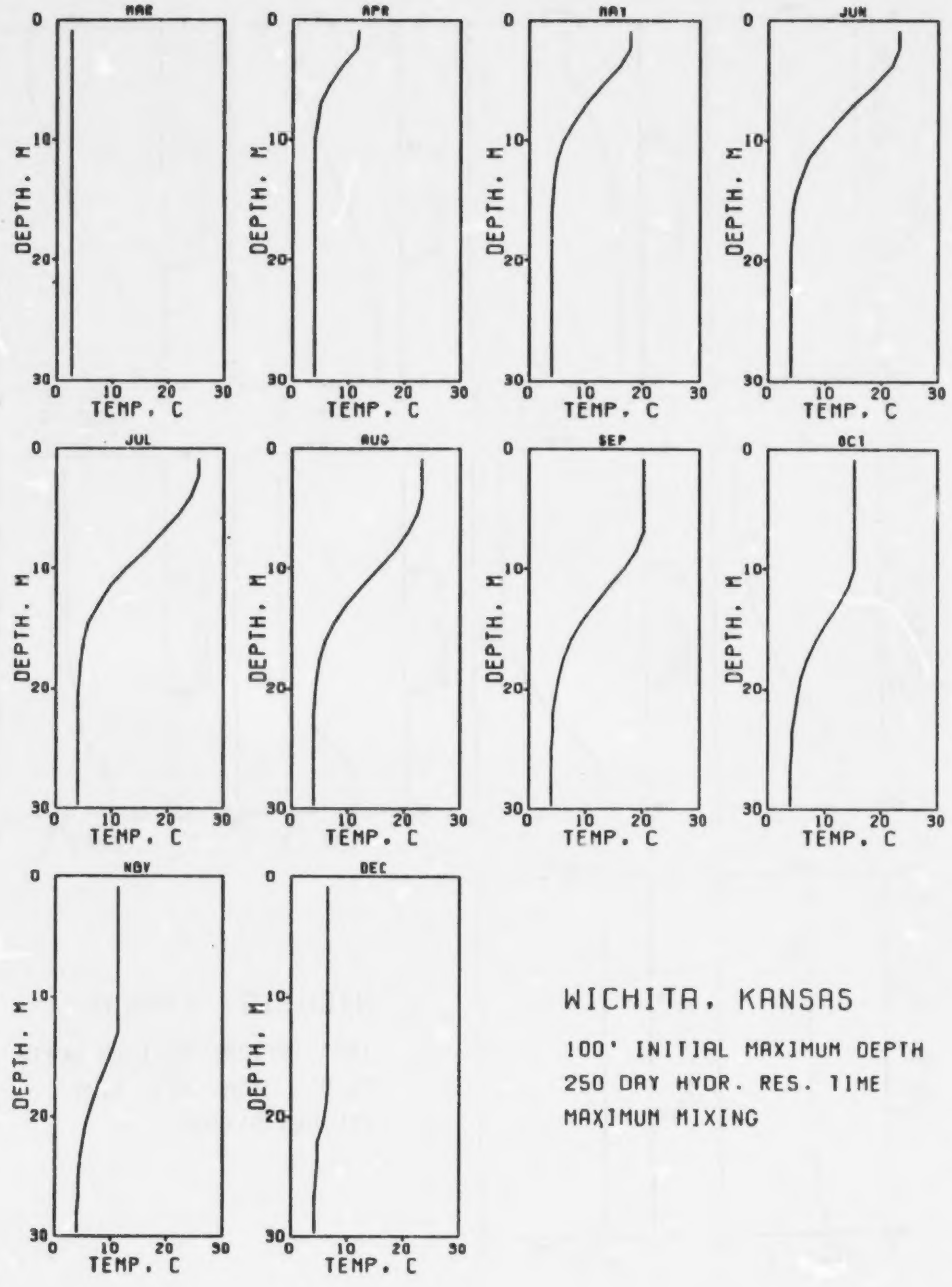
WICHITA, KANSAS

100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



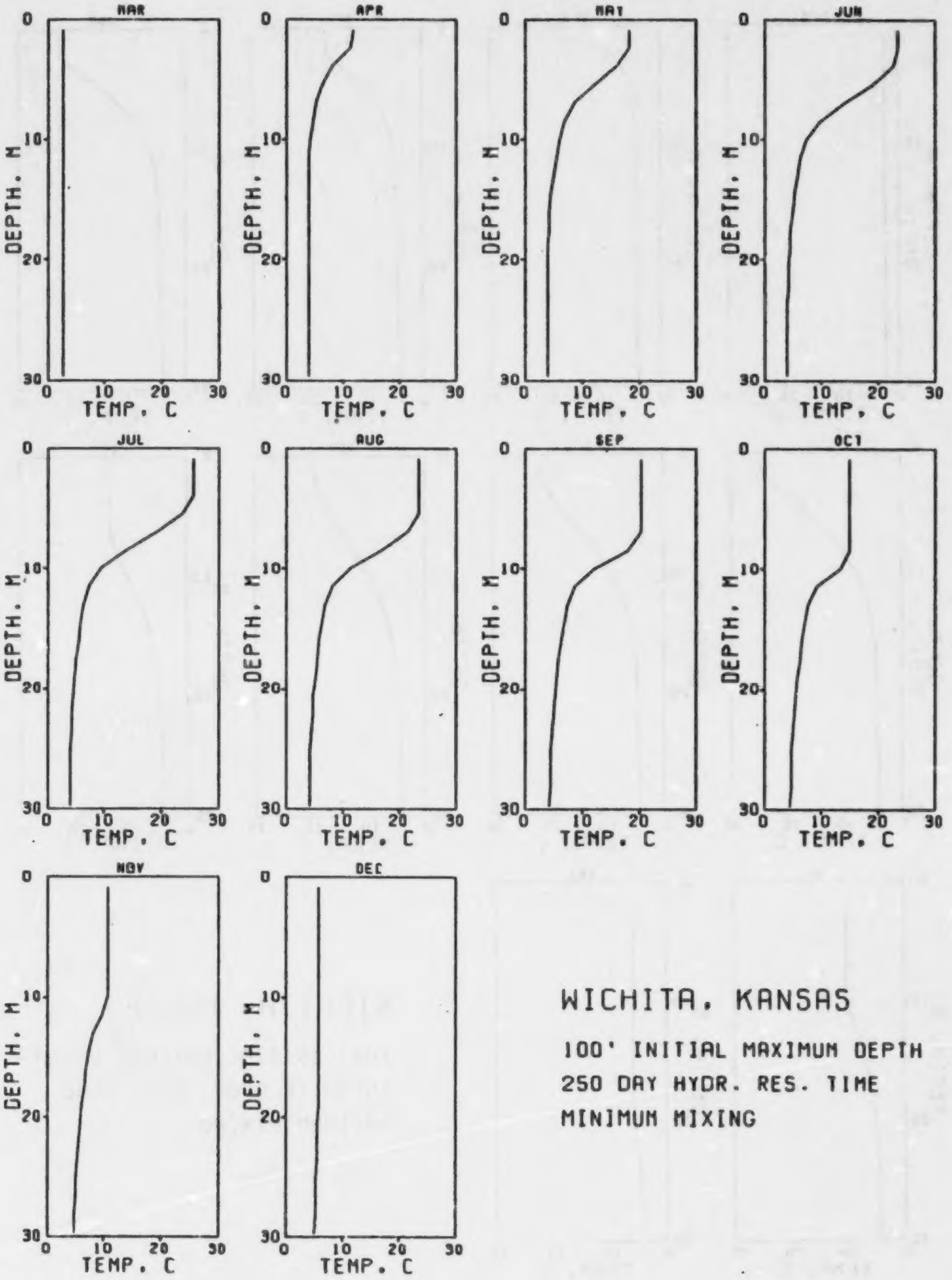
WICHITA, KANSAS

100' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



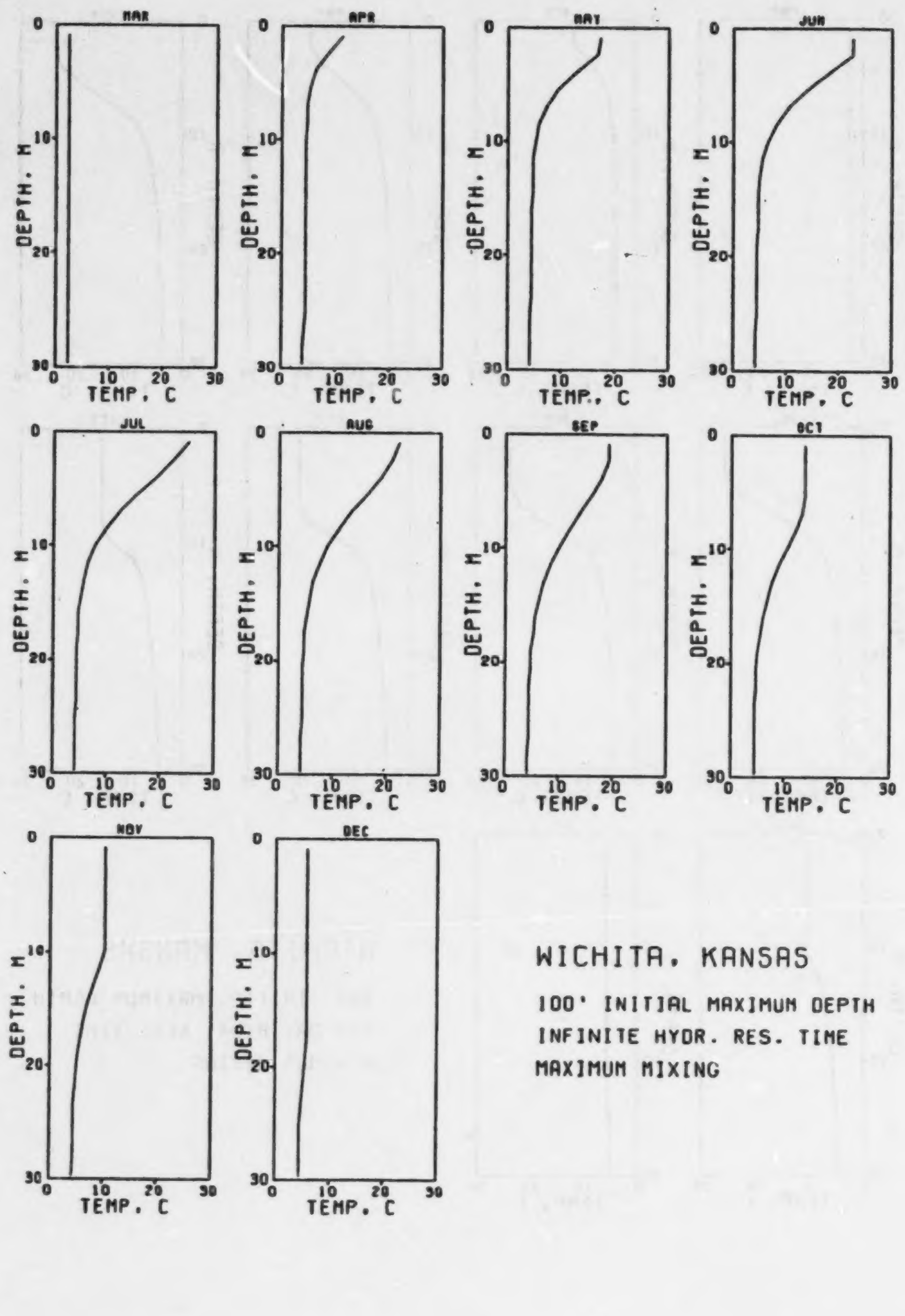
WICHITA, KANSAS

100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



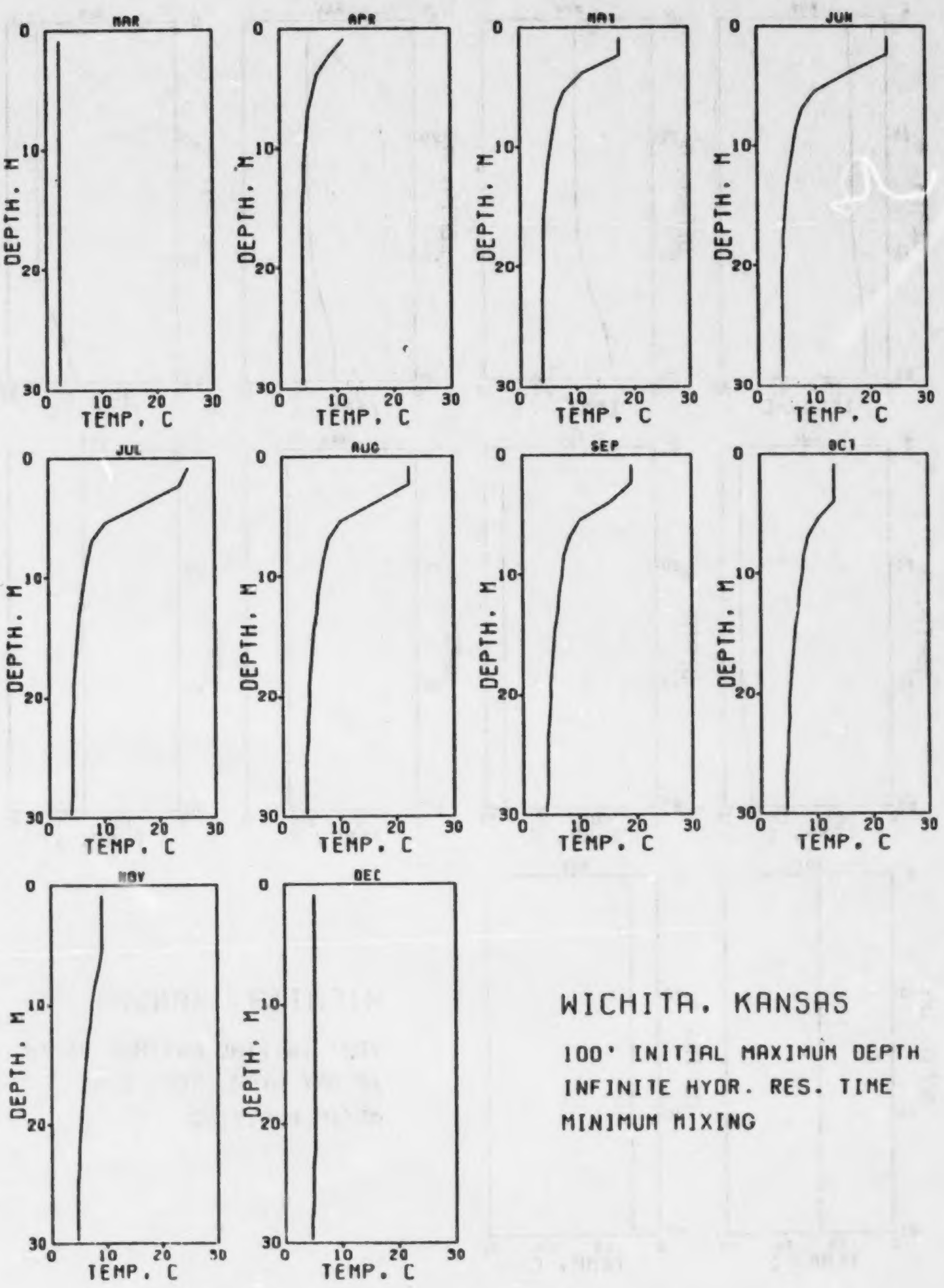
WICHITA, KANSAS

100' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



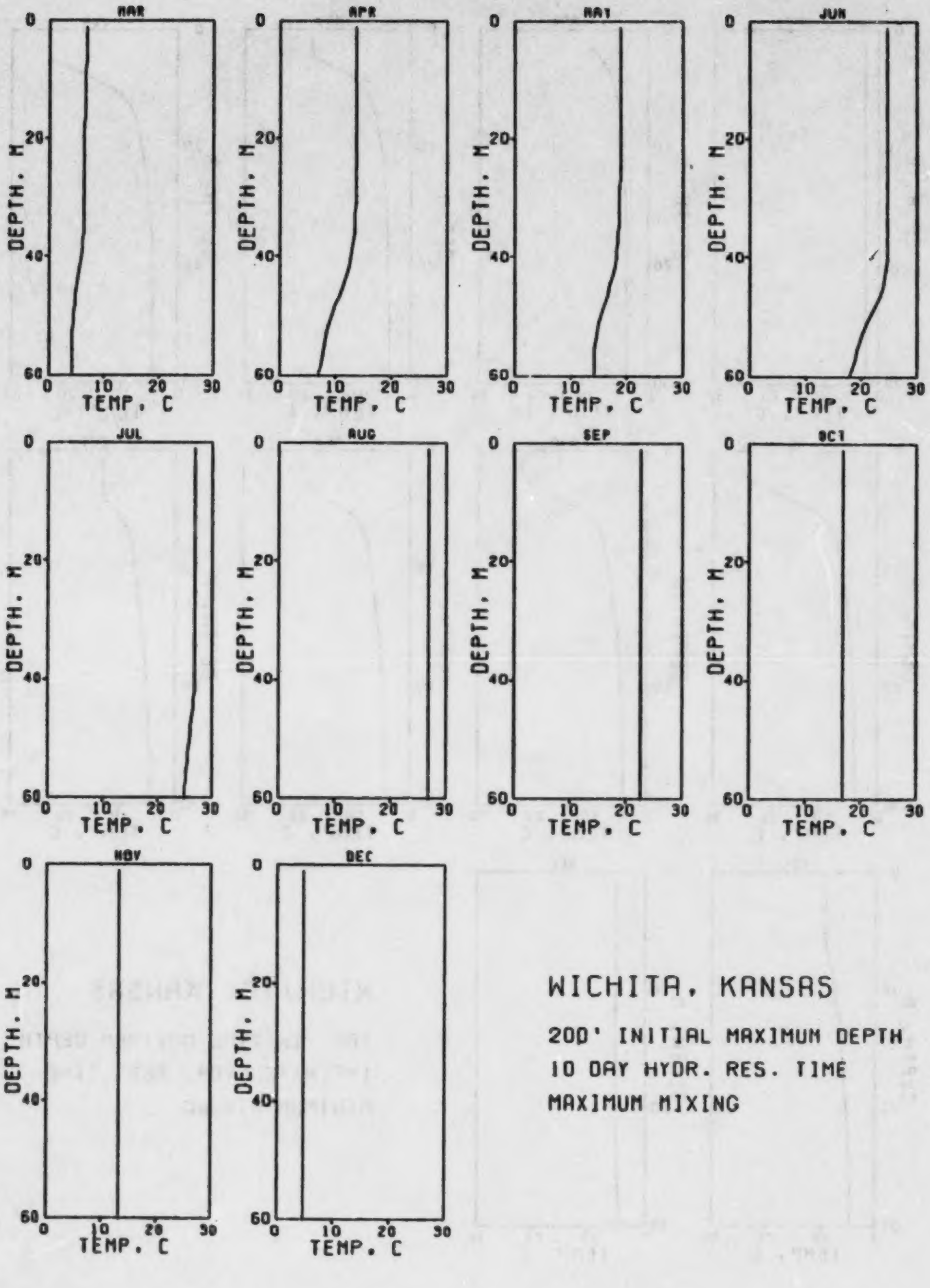
WICHITA, KANSAS

100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



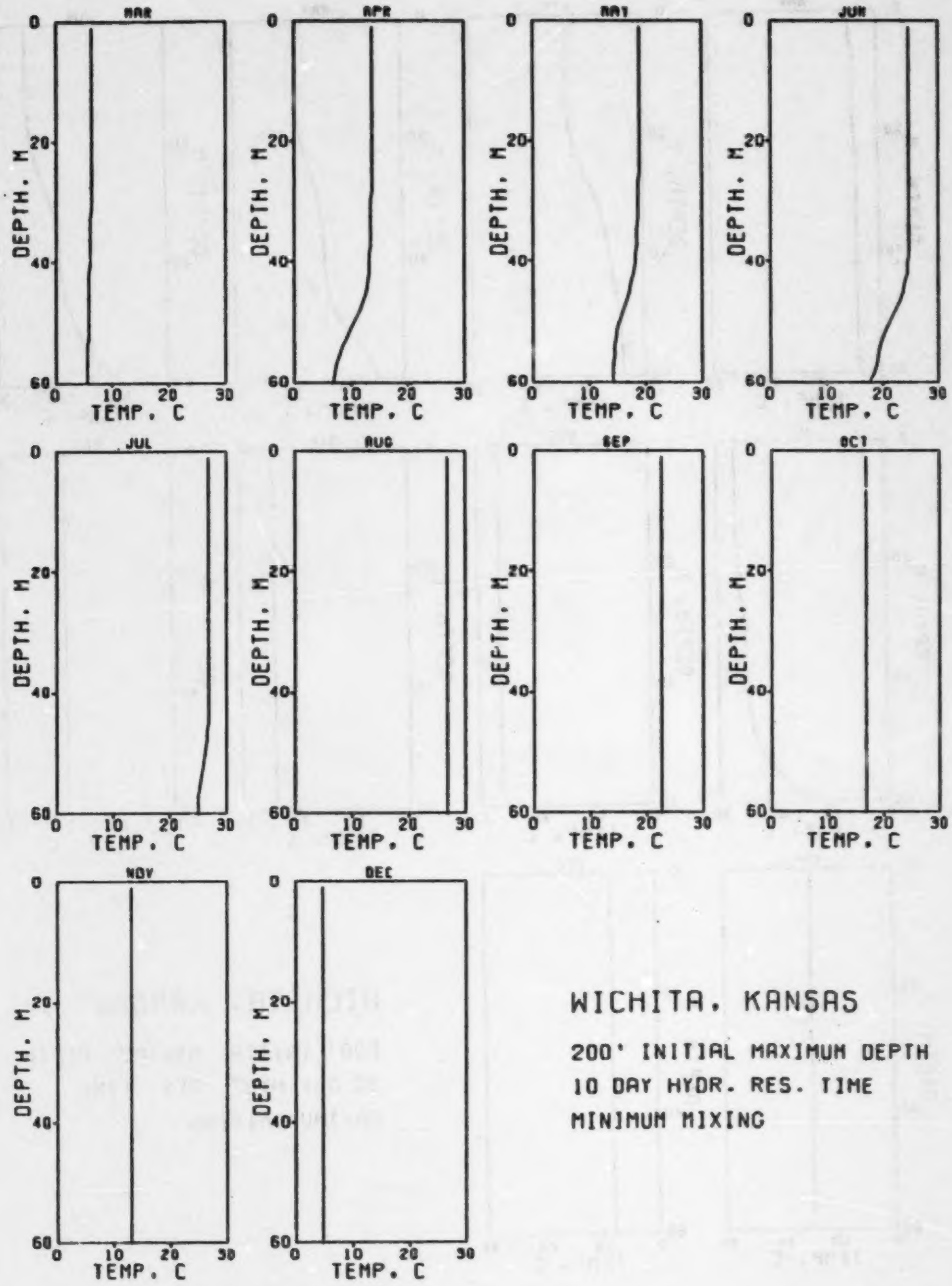
WICHITA, KANSAS

100' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING



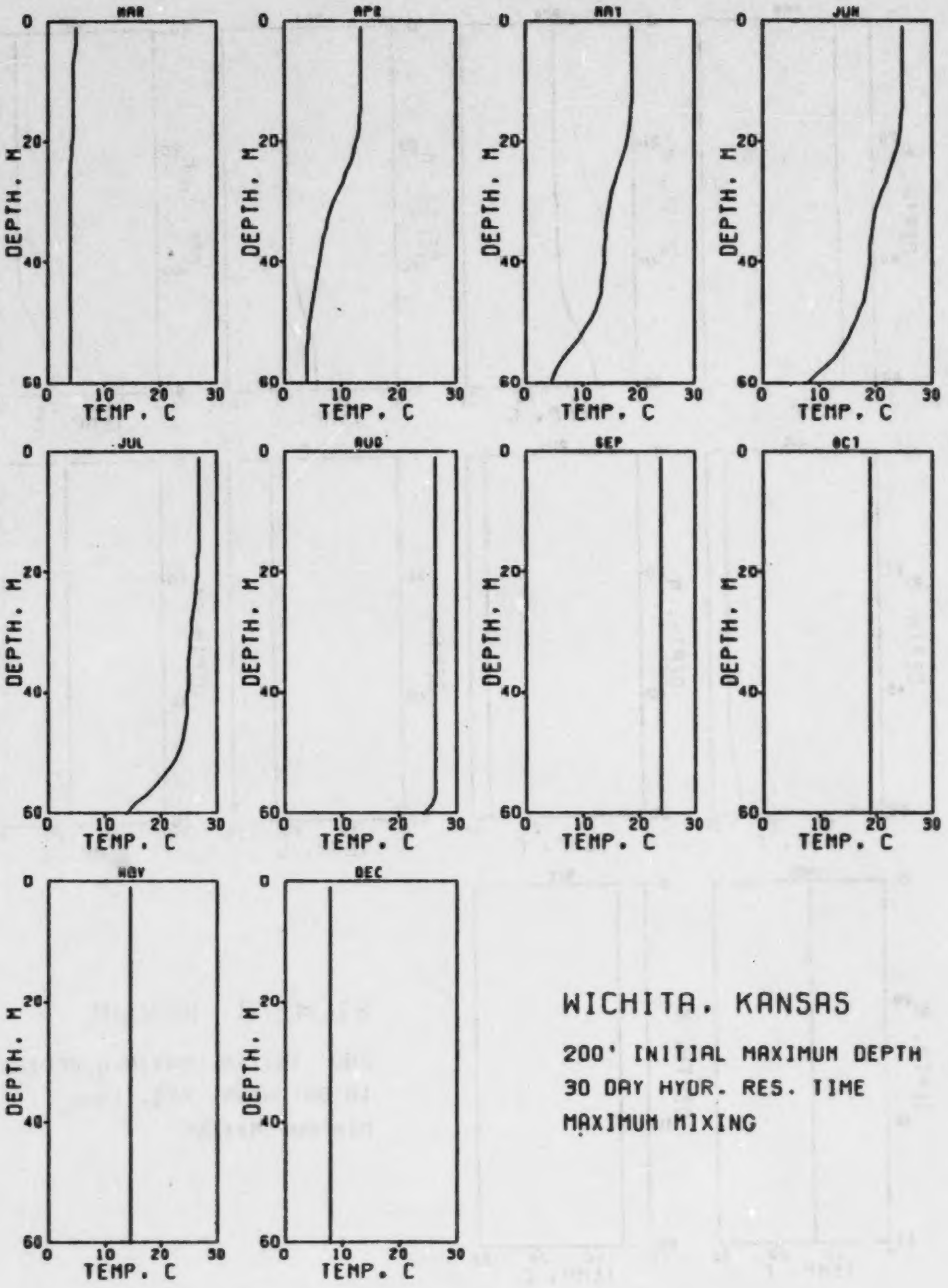
WICHITA, KANSAS

200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MAXIMUM MIXING



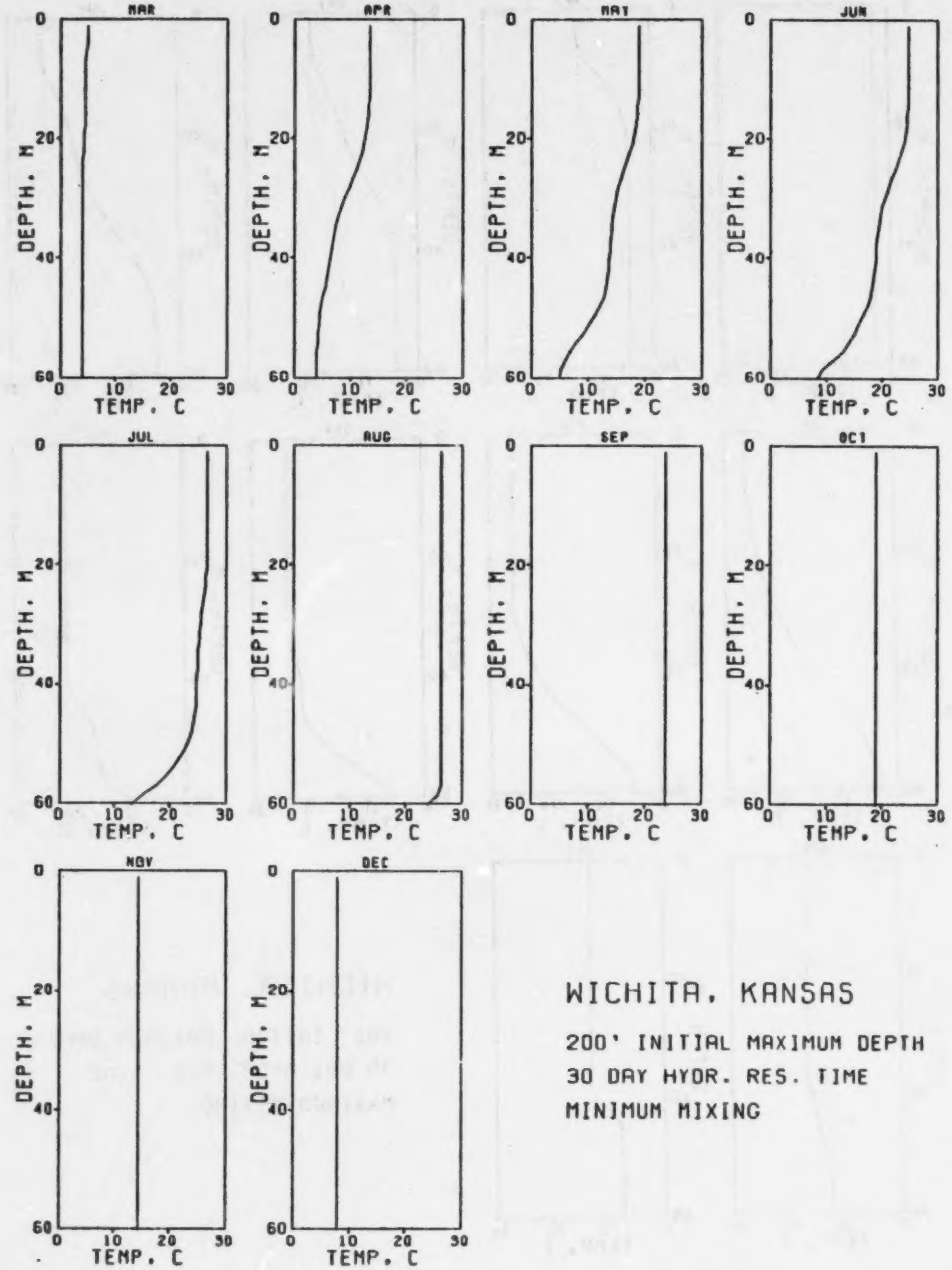
WICHITA, KANSAS

200' INITIAL MAXIMUM DEPTH
10 DAY HYDR. RES. TIME
MINIMUM MIXING



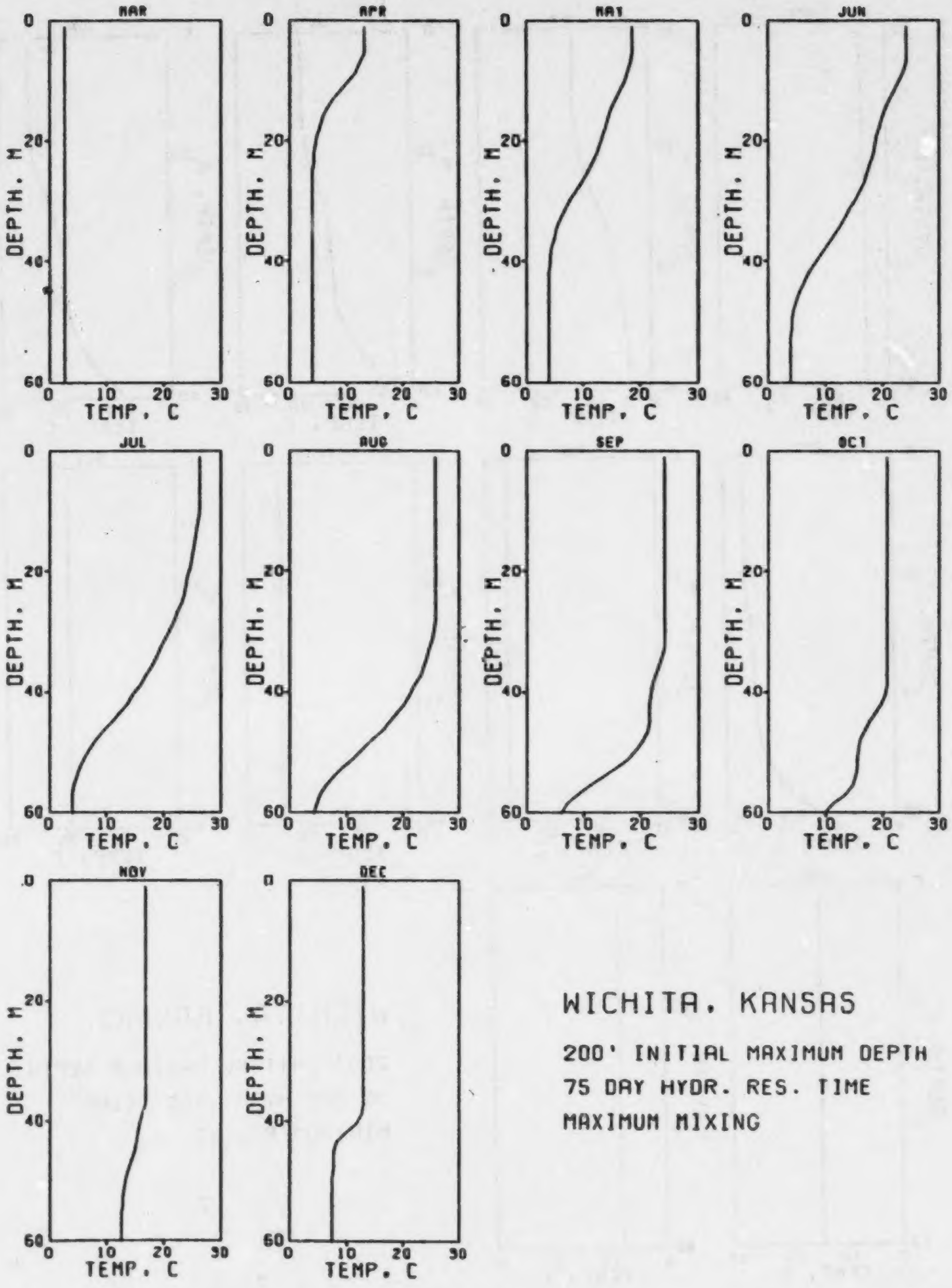
WICHITA, KANSAS

200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MAXIMUM MIXING



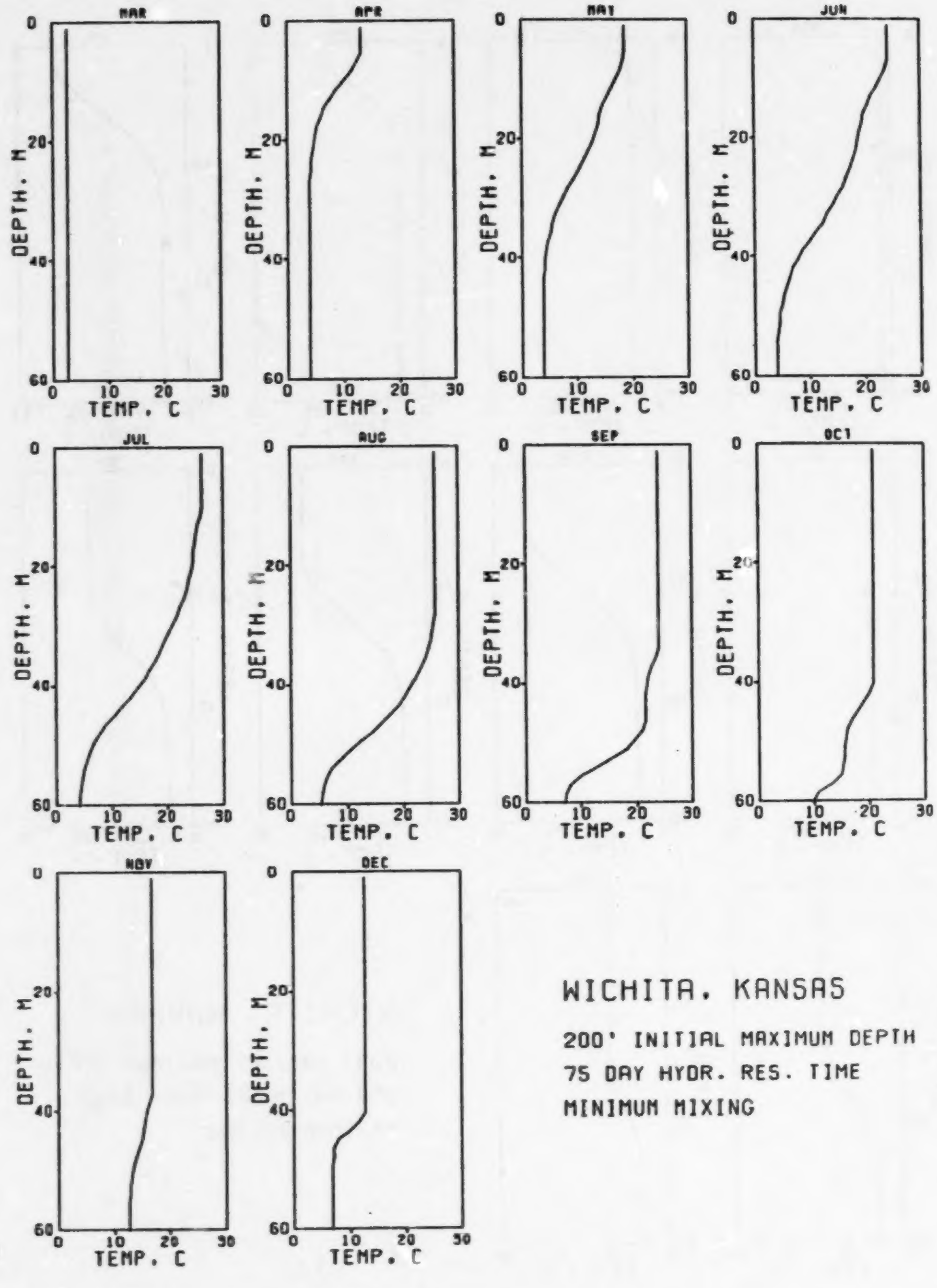
WICHITA, KANSAS

200' INITIAL MAXIMUM DEPTH
30 DAY HYDR. RES. TIME
MINIMUM MIXING



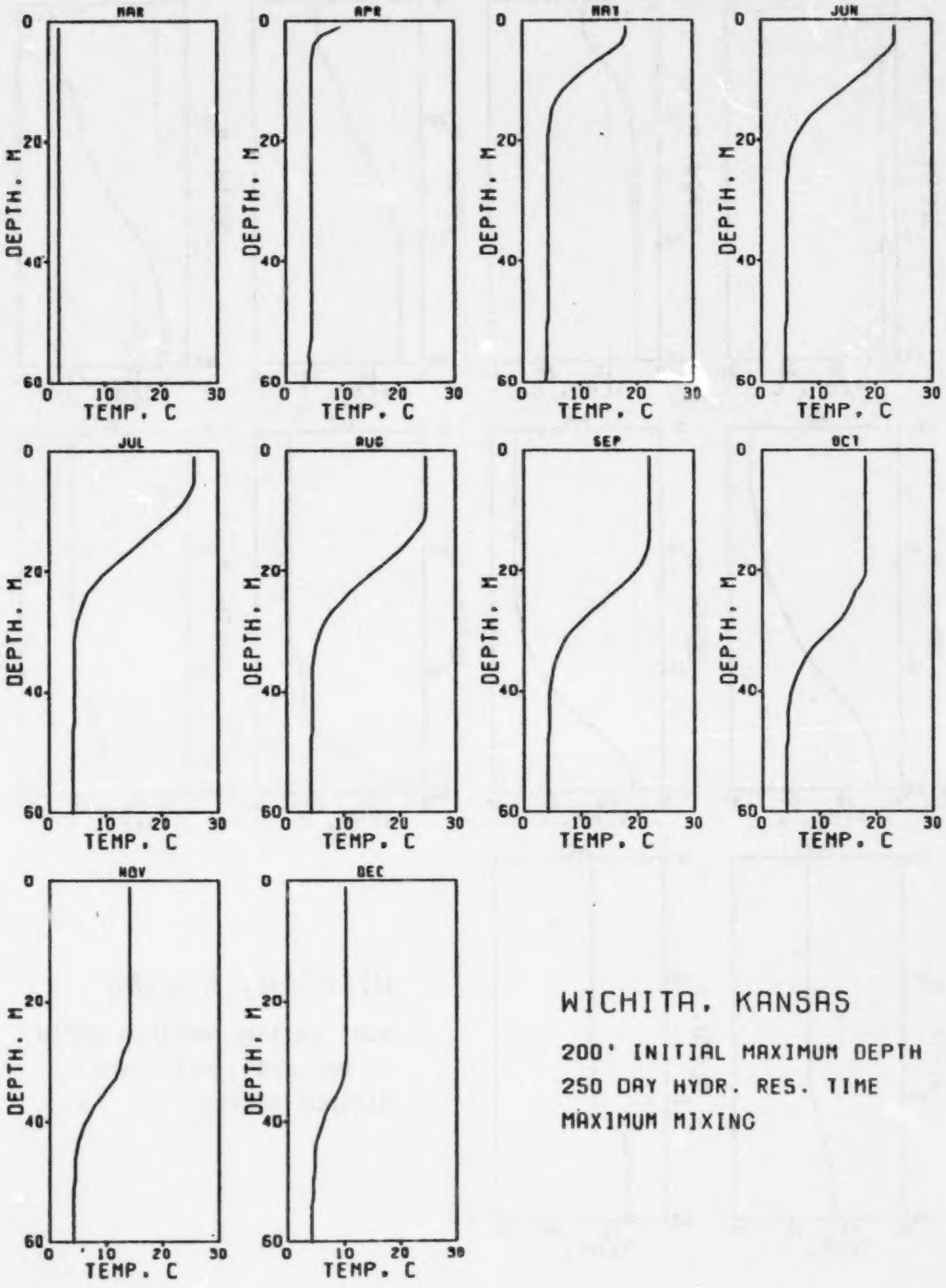
WICHITA, KANSAS

200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MAXIMUM MIXING



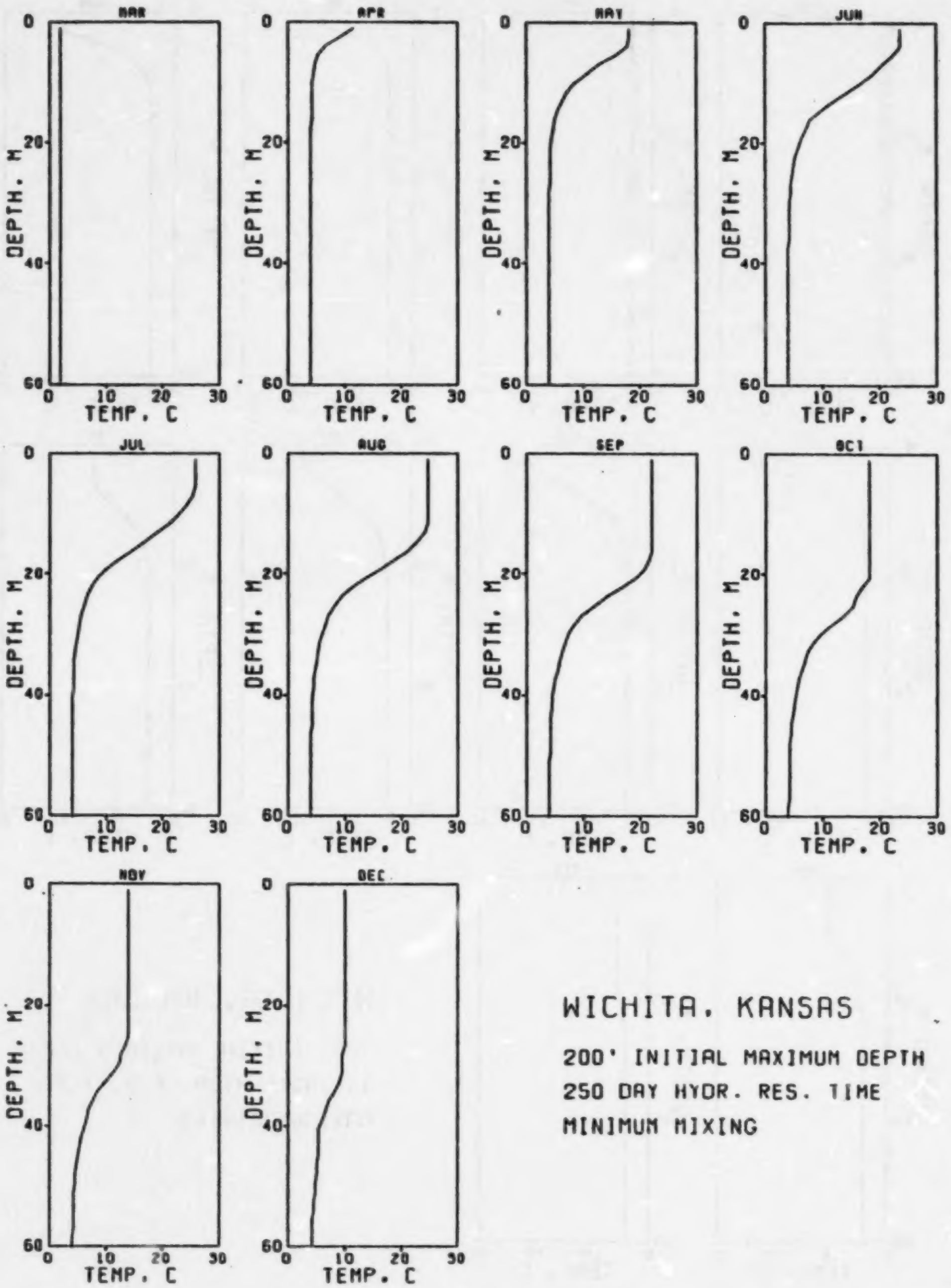
WICHITA, KANSAS

200' INITIAL MAXIMUM DEPTH
75 DAY HYDR. RES. TIME
MINIMUM MIXING



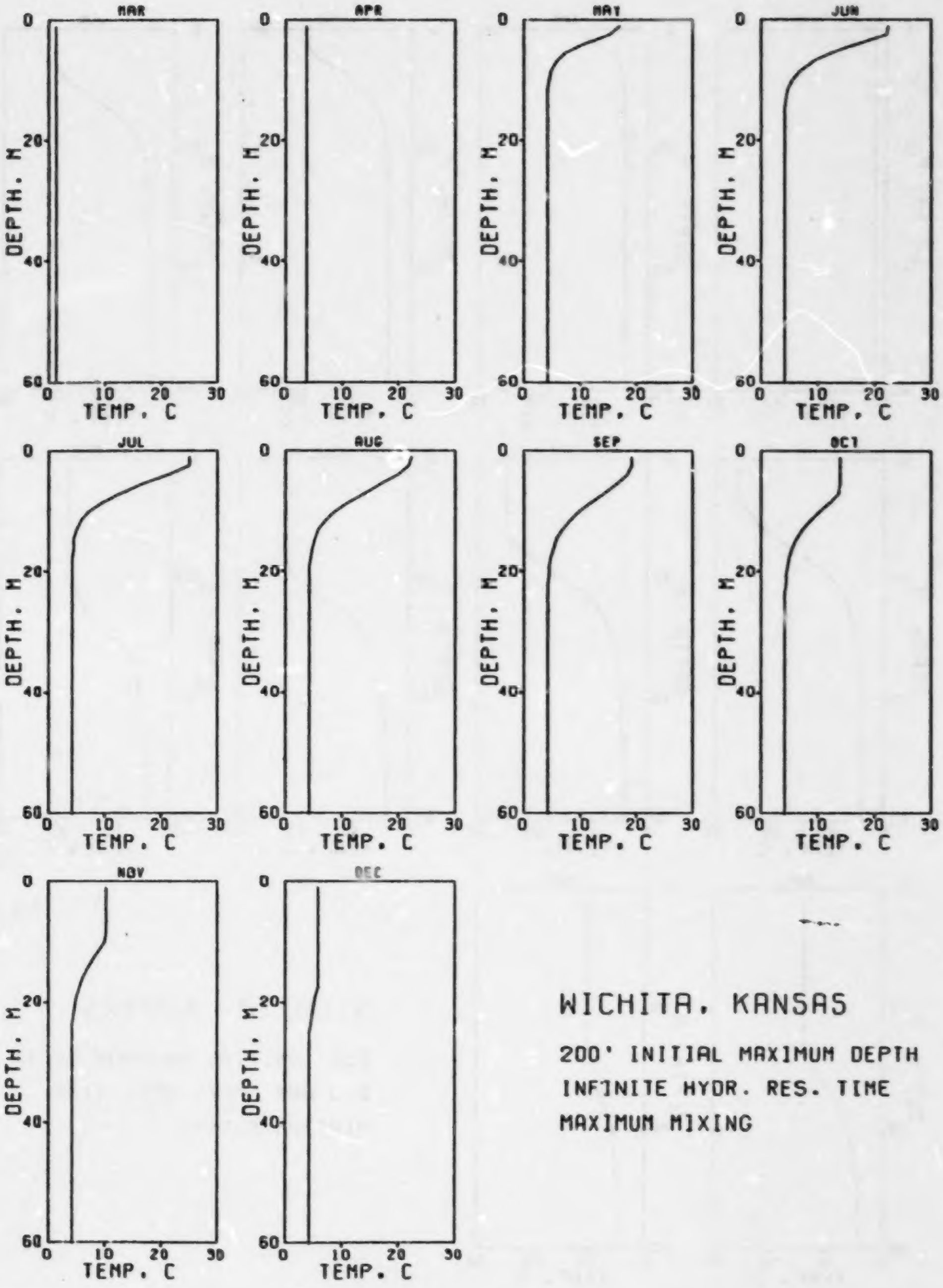
WICHITA, KANSAS

200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MAXIMUM MIXING



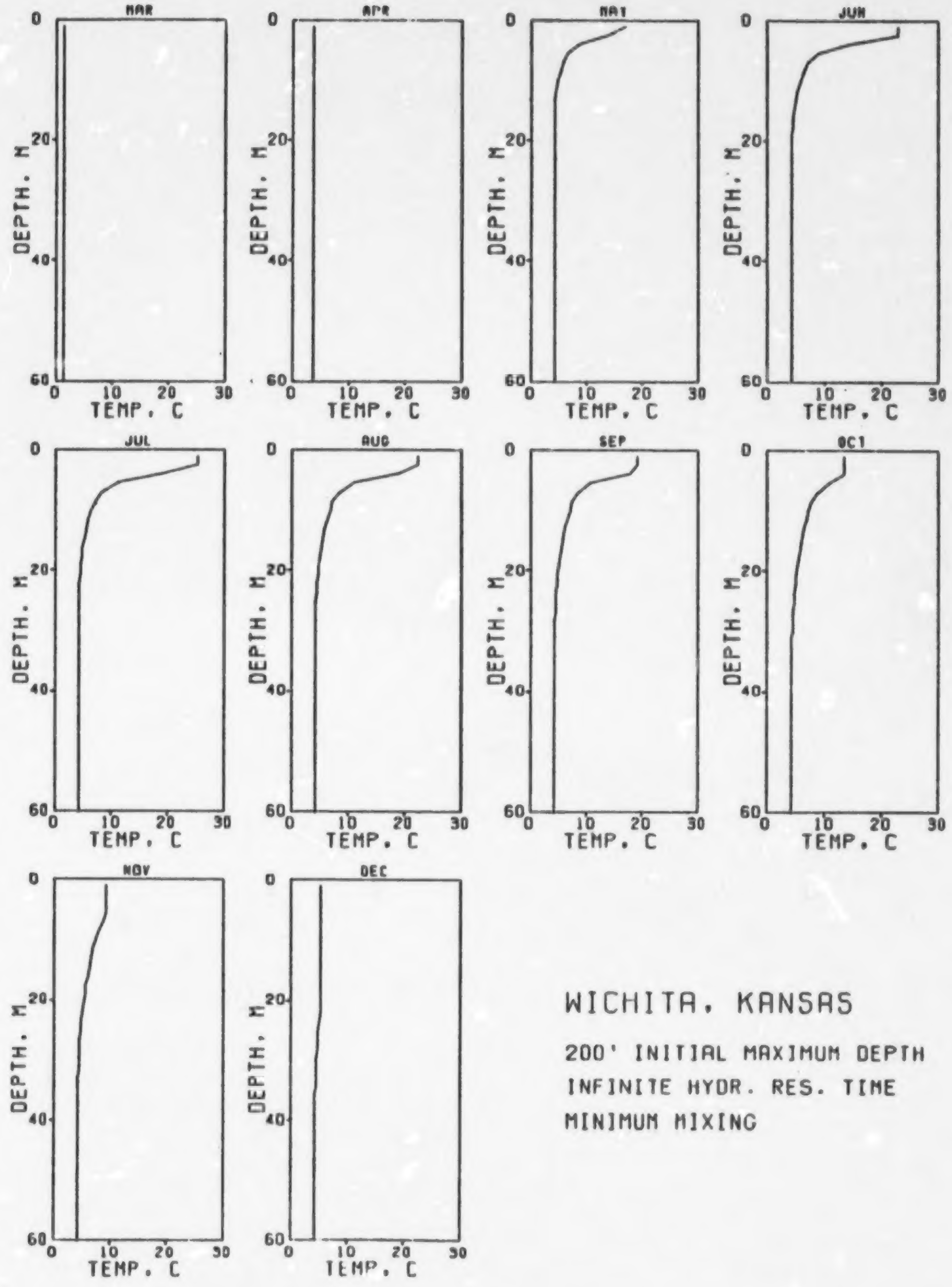
WICHITA, KANSAS

200' INITIAL MAXIMUM DEPTH
250 DAY HYDR. RES. TIME
MINIMUM MIXING



WICHITA, KANSAS

200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MAXIMUM MIXING



WICHITA, KANSAS

200' INITIAL MAXIMUM DEPTH
INFINITE HYDR. RES. TIME
MINIMUM MIXING

APPENDIX E

MODELING THERMAL STRATIFICATION IN IMPOUNDMENTS

Figure E-1 Comparison of Computed and Observed Temperature Profiles in Kezar Lake

Figure E-2 Comparison of Computed and Observed Temperature Profiles in El Capitan Reservoir

Figure E-3 Log of Eddy Conductivity Versus Log Stability--Hungry Horse Data

IMPOUNDMENT THERMAL PROFILE MODEL: BACKGROUND

The model used for computation of impoundment temperature profiles is based on the Lake Ecologic Model originally developed by Chen and Orlob (1975). The model was modified for this application to compute temperature alone. The purpose of the model application was to simulate the effects of mixing, impoundment physical characteristics, hydraulic residence time, and climate on the vertical profiles of temperature.

Physical Representation

Each configuration simulated was idealized as a number of horizontally mixed layers. Natural vertical mixing is computed by the use of dispersion coefficients in the vertical mass transport equation. Values of the dispersion coefficients for different size lakes were estimated from previous studies (Water Resources Engineers, Inc., 1969).

Temperature

Temperatures were computed as a function of depth according to Equation (E-1).

$$\bar{v}\frac{\partial T}{\partial t} = \frac{1}{c\rho}\frac{\partial}{\partial z}\left(A_z D_z \frac{\partial T}{\partial z}\right) - \frac{\partial}{\partial z}(QT) + \frac{A_s}{c\rho}(\mu - \lambda T) + \frac{\Theta}{c\rho} - T\frac{\partial \bar{v}}{\partial t} \qquad \text{(E-1)}$$

where T = the local water temperature
c = specific heat
ρ = fluid density
A_z = cross-sectional area at the fluid element boundary

t = time

z = vertical distance

D_z = the eddy diffusion coefficient in the vertical direction

Q = advection across the fluid element boundaries

A_s = cross-sectional area of the surface fluid element

μ, λ = coefficients describing heat transfer across air-water interface

Θ = sum of all external additions of heat to fluid volume of fluid element

$\overline{v}$ = element volume

Application/Verification

The model has recently been used in a lake aeration study (Lorenzen and Fast, 1976). In that study, the model was applied to Kezar Lake in New Hampshire and El Capitan Reservoir in California to verify that artificial mixing could be adequately simulated.

Computed temperature profiles were compared to observed values as shown in Figures E-1 and E-2. The model performance was judged to be good for the intended purpose of providing guidance for further study.

PREPARATION OF THERMAL PROFILES

The thermal profiles in Appendix D of this report were prepared by inputting the selected climatological conditions, inflow rate, impoundment physical conditions, and wind. Of these, only wind warrants special discussion here. The remaining model parameters are discussed in the text of Chapter 5.

Wind-Induced Mixing and the Eddy Diffusion Coefficient

Figure E-3 is a plot of the eddy conductivity coefficient versus stability. It was used to obtain coefficients for wind mixing for the

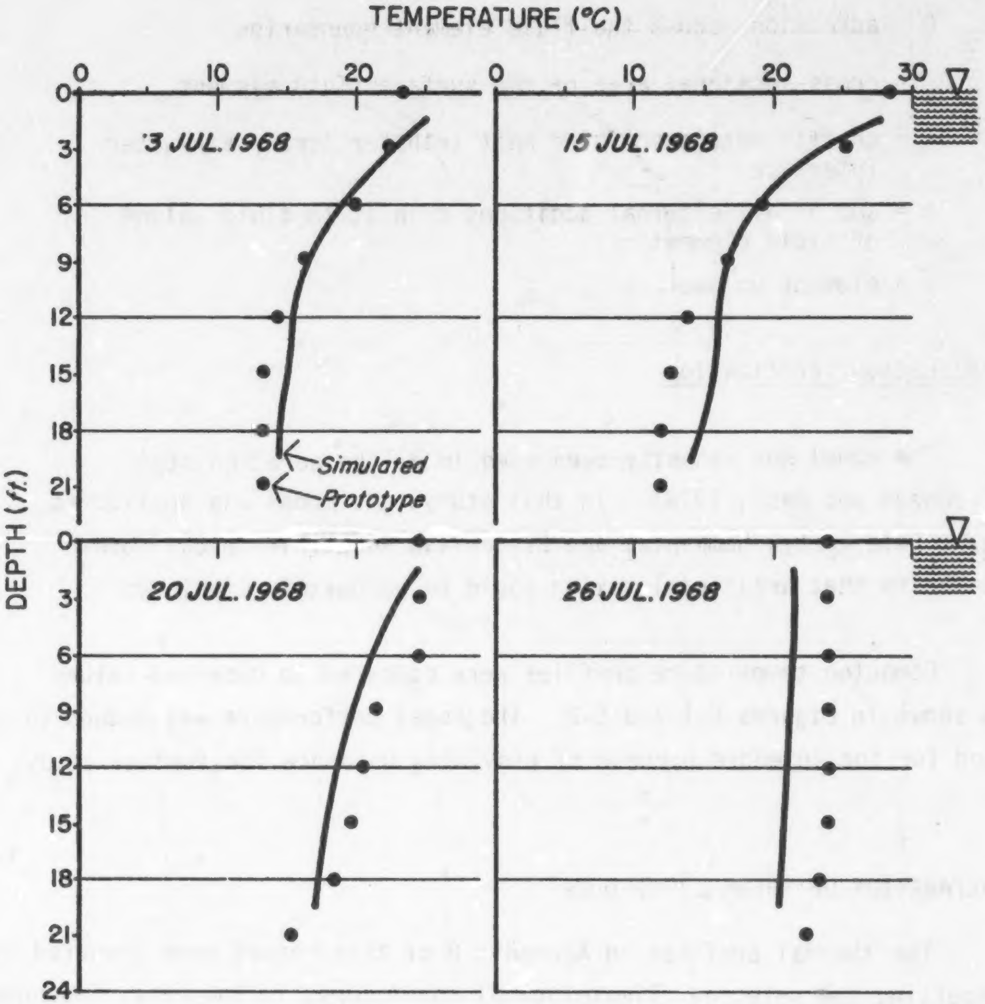


FIGURE E-1 COMPARISON OF COMPUTED AND OBSERVED TEMPERATURE PROFILES IN KEZAR LAKE

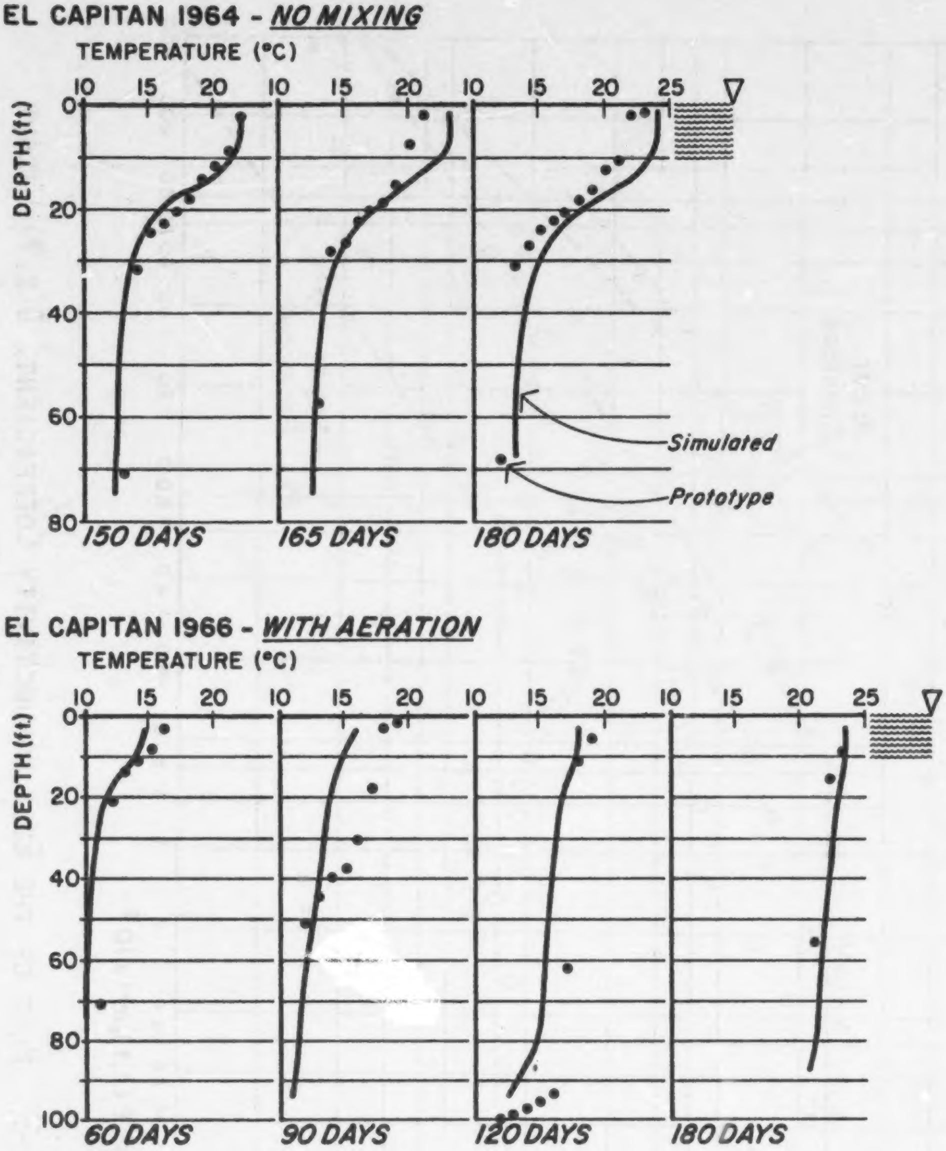


FIGURE E-2 COMPARISON OF COMPUTED AND OBSERVED TEMPERATURE PROFILES IN EL CAPITAN RESERVOIR

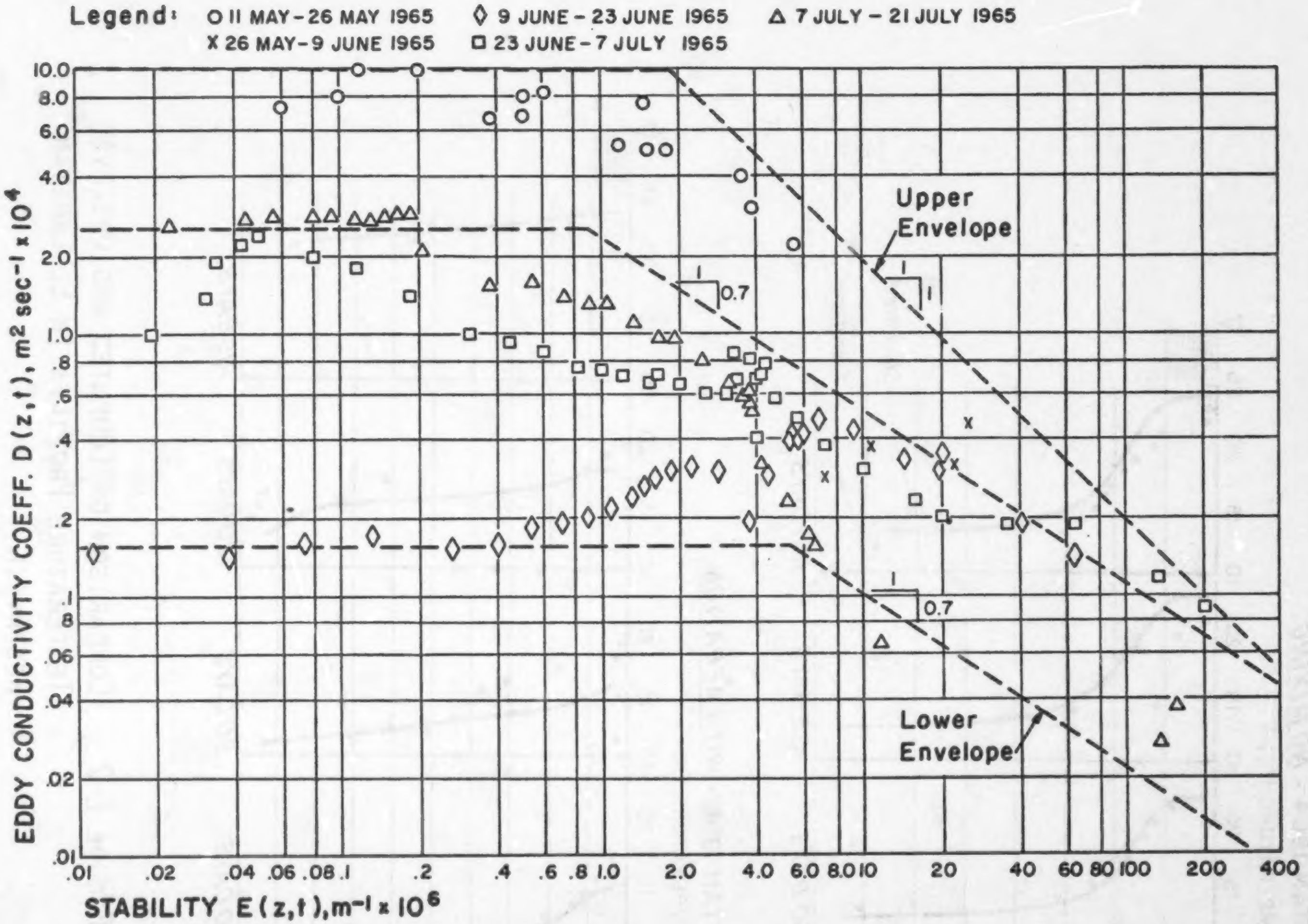


FIGURE E-3 PLOT OF THE EDDY CONDUCTIVITY COEFFICIENT, D(Z,T) VERSUS STABILITY, E(Z,T) FOR HUNGRY HORSE RESERVOIR DATA (AFTER WATER RESOURCES, INC., 1969) model runs. The upper envelope represents high wind mixing conditions and the lower envelope represents low wind mixing conditions. Note that the plot in Figure E-3 was developed for this model, and the model was then verified with data from Hungry Horse Reservoir, which is located on the South Fork of the Flathead River in northwestern Montana. Accordingly, the extremes of wind mixing and the effects on impoundment stability are as found for Hungry Horse Reservoir. The coefficients should be applicable elsewhere, however, because the eddy diffusion coefficient is relatively insensitive to climate and location.

The significance of the eddy conductivity coefficient and its implications for wind mixing may be understood by examining an equation describing transport within the system. Mixing implies the transfer of materials or properties within a system from points of high concentration to points of low concentration, and vice versa. For a system which is undergoing forced convection, it has been observed that the time rate of transport, F, of a property, S, through the system is proportional (other things being equal) to the rate of change of concentration of this property with distance, z. In equation form, this rule is expressed as:

$$F = - D \frac{\partial S}{\partial z} \qquad \text{(E-2)}$$

where D is the coefficient of proportionality. The mixing process as defined by Equation (E-2) is variously called "effective diffusion," "eddy diffusion," or the "diffusion analogy" because it is identical in form to the equation describing the process of molecular diffusion. The difference between the two processes, however, is that for molecular diffusion, D is constant, while for turbulent transfer, D is a function of the dynamic character, or the turbulence level, of the system. In general, D is a temporal and spatial variable, and thus will be referred to here as D(z,t). Equation (E-2) rewritten for heat flow over the reservoir vertical axis is

$$H = -\rho c D(z,t) \frac{\partial T}{\partial z} \qquad \text{(E-3)}$$

where H = heat flux, $HL^{-2}T^{-1}$
ρ = density of water, ML^{-3}
c = heat capacity of water, $HM^{-1}D^{-1}$
$D(z,t)$ = coefficient of eddy conductivity, L^2T^{-1}
T = temperature, D
z = elevation in the reservoir, L
t = time T

From Equation (E-3), therefore, it may be seen that the rate of heat flux (H), which describes the rate of energy transfer vertically in an impoundment, is a function of the temperature gradient over depth ($\frac{\partial T}{\partial z}$) and the degree of turbulence (induced by wind and other factors) and is characterized by the eddy diffusion coefficient D(z,t) in the equation. It is this coefficient, D(z,t) which is plotted on the ordinate (stability is on the abscissa) in Figure E-3.

NOTE:

For a more detailed description of the model, its applicability, and the eddy diffusion coefficient, the reader is referred to a report entitled "Mathematical Models for the Prediction of Thermal Energy Changes in Impoundments." (See the list of references at the end of this Appendix.)

REFERENCES FOR APPENDIX E

Chen, C.W., and Orlob, G.T., 1975. Ecologic simulation for aquatic environments in systems analysis and simulation in ecology. Academic Press, N.Y., San Francisco, and London. III:475-588.

Lorenzen, M.W., and Fast, A., 1976. A Guide to Aeration/Circulation Techniques for Lake Management: For U.S. Environmental Protection Agency Corvallis, Oregon.

Water Resources Engineers, Inc., 1969. Mathematical Models for the Prediction of Thermal Energy Changes in Impoundments. Water Quality Office, Environmental Protection Agency.

APPENDIX F

RESERVOIR SEDIMENT DEPOSITION SURVEYS

The material in this appendix consists of a reproduction of a bulletin compiled by F. E. Dendy and W. A. Champion, which provides data on rates of sedimentation in U. S. reservoirs.

INTRODUCTION

Data from known reliable reservoir sedimentation surveys made in the United States through 1970 are summarized in this bulletin. Additional data from surveys made after 1970 are included for a few reservoirs.

This bulletin supersedes USDA Miscellaneous Publication No. 1143, which was published in May, 1969.[1/] All reservoir surveys reported in Miscellaneous Publication No. 1143 have been repeated in this bulletin. In addition, it includes surveys made before 1965, but not previously reported, and new data on reservoirs surveyed or resurveyed since 1965. The reservoirs are located in all of the 48 conterminous United States, except Florida, and in Puerto Rico. In addition to data on storage reservoirs and ponds, some information on debris basins is included.

A supplement to this bulletin, from which the data were extracted and summarized, contains detailed information about each of the reservoirs

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[1/] Dendy, F.E. and Champion, W.A., Compilers. Summary of Reservoir Sediment Deposition Surveys Made in the United States Through 1965. U.S. Department of Agriculture Miscellaneous Publication No. 1143, 64 pp., May, 1969. (Cooperative report with the Sedimentation Committee. Water Resources Council).

listed in the summary table. The method used in presenting this information is given on pages F-2, F-3, F-4 and F-5. The supplement has not been distributed with this bulletin because of its bulk and because the detailed information is not of general interest. Copies are available in the offices of the agencies represented on the Sedimentation Committee of the Water Resources Council. Reprints of data sheets for specific reservoirs may be obtained on request from the Director, USDA Sedimentation Laboratory, U.S. Department of Agriculture, Oxford, Miss. 38655. Requests for information not contained in this bulletin or in the supplement should be directed to the agency supplying the data.

The accuracy of the survey data varies greatly. Surveys range from reconnaissance measurements of sediment depth at a few locations to detailed surveys based on closely spaced cross sections or contours. No attempt has been made to classify the surveys according to degree of accuracy.

Information in this bulletin and in the supplement should prove useful to engineers and watershed planning specialists in private and public practice who are concerned with problems of reservoir sedimentation. Engineers, engineering firms and local government agencies who have data on similar reservoir surveys are invited to make this information available to the Sedimentation Committee, WRC, for inclusion in supplements to this publication.

EXPLANATION OF THE SUMMARY TABLE

Data in the summary table of this bulletin were obtained from the reservoir sedimentation survey data sheets contained in the supplement. Dashes in columns of the table signify that data were unavailable or that the column is not applicable for the reservoir.

Reservoirs are grouped according to the 79 drainage areas into which the United States has been divided as shown in the publication, "River Basin Maps Showing Hydrologic Stations," compiled under the auspices of the Subcommittee on Hydrology, Federal Inter-Agency River Basin

Committee.[2/] An index map of these drainage areas is shown on page F-78. The drainage areas in which the reservoirs are located are shown as subheadings in the summary table. The first of the two numbers identifying a reservoir indicates the drainage basin in which it is located. The second number denotes the particular reservoir in the drainage area and is based upon the order in which the data were prepared. These numbers are the same as those identifying the corresponding survey data sheets in the supplement. When a survey data sheet is revised or when another sheet is prepared with information for additional surveys, the identification number is modified by the addition of letters beginning with a; for example, 13-2, 13-2a, and 13-2b.

Total drainage area includes the reservoir area and the area lying above all upstream dams but generally excludes noncontributing drainage areas lying within the watershed boundary. Where available, the drainage area figure published by the U.S. Geological Survey in Water-Supply Papers is usually used. The net drainage area is the sediment-contributing area and generally excludes the reservoir area and the drainage areas above upstream reservoirs, or other structures which are effective sediment traps.

The first date shown usually corresponds to the beginning of storage when sediment deposition began. However, for some reservoirs the first date represents the date of the contour or range survey made after the reservoir had been in operation for some time.

For most reservoirs, the storage capacity given is the total storage below the level of the crest of an ungated spillway or the top of gates (less gate-height freeboard, if any) of gated spillways. Where capacity values below the spillway crest elevation are given, footnotes are used to explain.

[2/] U.S. Inter-Agency Committee on Water Resources, Subcommittee on Hydrology. River Basin Maps Showing Hydrologic Stations. U.S. Dept. Com., Weather Bur., Notes on Hydrol. Activ. Bul. 11, 79 pp., 1961.

The capacity-average annual inflow ratio (C/I ratio) was derived from the reservoir storage capacity and the average annual inflow. Normally the average annual inflow for the entire period of record was used to compute the C/I ratios. This time period may or may not correspond to the period for which sediment accumulation was given. Generally, the C/I ratio was not given if upstream structures controlled 25 percent or more of the drainage area.

The specific weight of deposited sediment is an average or weighted value for the reservoir, determined generally from samples of deposits. In view of the variations with depth and location within the reservoir, specific weight is generally an approximation for the reservoir. The entry is marked by an asterisk where the specific weight is assumed or is calculated from field data or the size-frequency grading of the deposits.

The average annual rate of sediment accumulation (acre-feet and tons per square mile of net drainage area) pertains to sediment deposited in the reservoir below the full pool elevation. Sediment deposited in deltas above full pool level or sediment discharged from the reservoir is not included unless explained by footnote. For reservoirs with more than one survey and where the latest survey indicated an increase in the specific weight of deposited sediment, the annual sediment accumulation rate in tons per square mile was not always computed in the same manner. For some reservoirs, compaction of earlier sediment was considered and in others it was not. All of the deposited sediment was assumed to have been transported into the reservoir by water.

The agency supplying data is shown in the last column of the table. This agency either has the basic data available or has access to it through cooperative arrangements. The symbols used in this column apply to the following agencies:

ARS - Agricultural Research Service
BR - Bureau of Reclamation
CE - Corps of Engineers
FS - Forest Service
GS - Geological Survey
IWS - Illinois State Water Survey
ODW - Ohio Department Natural Resources--Division of Water
SCS - Soil Conservation Service
TVA - Tennessee Valley Authority

FORM FOR REPORTING RESERVOIR SEDIMENTATION

A completed sample of the reservoir sedimentation data sheet from the supplement is shown on pages F-79 and F-80. This sheet is a convenient and standard form for reporting results of reservoir surveys. An invitation is extended to readers, particularly those practicing engineering individually, in engineering firms, or in local government agencies, to prepare sheets covering surveys known to them but not included in this publication. A blank "Reservoir Sedimentation Data" sheet is enclosed as a tear sheet on pages F-81 and F-82. Additional data sheets may be obtained from the department offices listed on the title page or the form may be reproduced if desired. The completed forms may be sent to any one of the agencies represented on the Sedimentation Committee for inclusion in supplements to this bulletin.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
ST. JOHN MACHIAS, PENOBSCOT, KENNEBEC, ANDROSCOGGIN, AND PRESUMPSCOT RIVER BASIN													
1-1	Little River Res. #2	Little River	Belfast, Me.	13.9	13.8	— 1913	—	456	*0.029	—	—	--	SCS
	do	do	do	—	—	Oct. 1969	56	341	.021	*100	0.15	326	
HOUSATONIC, CONNECTICUT, THAMES, AND MERRIMACK RIVER BASIN													
2-1	Broad Brook	Broad Brook River	Broad Brook, Conn.	5.13	—	-- 1916	—	3,706	—	--	--	—	SCS
	do	do	do	—	—	Sept. 1951	35	3,652	—	*50	.33	359.4	
2-2	Mountain Street	Trib. of Beaver Brook	Greenfield, Mass.	.8	—	— 1905	—	1,076.2	—	—	--	—	SCS
	do	do	do	—	—	Sept. 1951	46	1,056.1	—	*50	.56	609.8	
2-3	Plants Pond	Eightmile Run	Plantsville, Conn.	15.76	—	Nov. 1850	—	105.0	—	—	--	—	SCS
	do	do	do	—	—	Sept. 1951	101	84.8	—	*50	.01	10.9	
2-4	Southington Reservoir	Budd River	Southington, Conn.	1.08	—	— 1883	—	321.8	—	—	--	--	SCS
	do	do	do	—	—	Sept. 1951	68	314.0	—	*50	.10	108.9	
2-5	Wallingford	Pine River	Wallingford, Conn.	8.98	—	— 1941	--	433.4	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1951	10	430.0	—	*50	.04	43.6	
2-6	West Whately	Avery Brook	West Whately, Mass.	9.0	—	-- 1902	—	76.7	--	—	—	—	SCS
	do	do	do	—	—	Sept. 1951	49	71.2	--	*50	.012	13.07	
2-7	Westfield	Moose Mountain Branch	Westfield, Mass.	2.52	—	— 1874	—	564.6	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1951	77	546.8	--	*50	.094	102.4	
HUDSON RIVER BASIN AND ST. LAWRENCE DRAINAGE IN NEW YORK													
3-1	Schoharie (Gilboa Dam)	Schoharie Creek	Prattsville, N. Y.	314	312.32	Aug. 1926	—	63,821	—	—	—	--	SCS
	do	do	do	—	—	May 1950	23.8	62,702	—	*60	1/.166	1/217	
3-2	Englishtown Pond	Weamaconk Creek	Englishtown, N. J.	7.2	7.19	— 1755	—	42.3	.005	—	—	—	SCS
	do	do	do	—	—	Oct. 1955	200	13.5	.002	*60	.02	26.14	
3-3	Carnegie Lake	Millstone River	Princeton, N. J.	155	47.8	Dec. 1907	—	1,256	—	—	--	—	SCS
	do	do	do	—	—	Sept. 1950	42.8	934	--	*50	2/.20	217.8	
	do	do	do	—	—	— 1959	9.0	3/822	--	—	—	—	
3-4	Wappinger	Wappinger Creek	Wappinger Falls, N. Y.	197	—	— 1900	—	604.4	—	—	—	—	SCS
	do	do	do	—	—	July 1952	52	461.6	—	*50	.01	10.89	
SUSQUEHANNA AND DELAWARE RIVER BASINS													
4-1a	Loch Raven Reservoir	Gunpowder Falls River	Towson, Md.	303	4/219.4	— 1914	—	5/70,169	—	—	--	—	SCS
	do	do	do	—	—	Oct. 1943	29	5/64,813	—	*60	4/.618	4/808	
	do	do	do	—	—	June 1961	18	64,072	—	57.3	.187	233	
4-2a	Prettyboy	do	Herford, Md.	80	77.5	Apr. 1933	—	60,979	—	—	—	—	SCS
	do	do	do	—	—	Oct. 1943	10.5	5/60,410	—	*60	.699	913	
	do	do	do	—	—	Sept. 1961	18	59,864	—	64.2	.391	547	
4-3	Griffin	Griffin Creek (Legget)	Scranton, Pa.	3.21	3.04	— 1888	—	1,991	—	—	—	—	SCS
	do	do	do	—	—	May 1941	53	1,953	--	—	.237	--	
4-4	Elmhurst	Roaring Brook	do	34.85	34.55	— 1890	—	3,746	--	—	--	--	SCS
	do	do	do	—	—	May 1941	51	3,606	--	—	.034	—	
4-5	Lake Williams	Cordorus Creek	York, Pa.	42.9	42.6	— 1912	—	2,686	—	—	—	—	SCS
	do	do	do	—	—	Apr. 1939	27	2,232	--	49.1	.394	421	
4-6	Williams Bridge	Stafford Meadow Brook	Scranton, Pa.	5.0	4.94	— 1893	—	1,051	—	—	—	--	SCS
	do	do	do	—	—	May 1941	48	1,016	—	—	.022	—	
4-7a	Atkisson Reservoir	Winter's Run	Bel Air, Md.	45.46	45.35	-- 1942	--	896.19	.025	--	—	—	SCS
	do	do	do	—	—	— 1954	12	705.32	.019	*60	.351	459	
	do	do	do	—	--	— 1965	11	617.06	.017	—	.177	231	
4-8a	Palto Dam	Dean Creek	Spencer, N. Y.	.41	.41	July 1955	—	56.04	.130	--	—	—	SCS
	do	do	do	—	--	Aug. 1957	2	55.90	.129	*80	.170	296	
	do	do	do	—	—	June 1966	8.9	55.60	.128	*79.2	.083	141	
4-9a	Pylkas Dam	do	do	.71	.71	July 1955	—	187.31	.250	—	--	—	SCS
	do	do	do	—	—	Aug. 1957	2	187.13	.250	*80	.130	226.5	
	do	do	do	—	—	June 1966	8.9	186.70	.249	*80	.066	119	
4-10	Columbia Dam	Paulins Kill River	Columbia, N. J.	171	171	July 1937	—	311.77	—	—	--	--	SCS
	do	do	do	—	—	July 1957	20	244.19	—	*64	.02	28	

4-11	Old Glatfelter	W. Br. Codorus Creek	Spring Grove, Pa.	74.3	—	—	1884	—	147.2	—	—	—	—	SCS
	do	do	do	—	—	Apr.	1939	55	10.0	—	—	.034	—	
4-12	Palington Reservoir	Powder Creek	do	2.91	2.90	—	1937	—	63.0	—	—	—	—	SCS
	do	do	do	—	—	Apr.	1939	1.6	59.9	—	33.2	.669	483	
4-13	Lake Rushford	Caneadea Creek	Caneadea, N. Y.	60.7	—	—	1925	—	28,000	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1951	26	27,426	—	*60	.37	484	
4-14	Icedale	Brandywine W. Br.	Coatesville, Pa.	20.0	—	—	1900	—	137	—	—	—	—	SCS
	do	do	do	—	—	July	1951	51	106	—	*50	.03	33	
4-15	Coatesville	Rock Run	do	5.0	—	—	1916	—	1,019	—	—	—	—	SCS
	do	do	do	—	—	July	1951	35	970	—	*50	.28	305	
4-16	Liberty Reservoir	Patapsco	Wards Chapel, Md.	164	159.1	July	1954	—	138,762.4	1.133	—	—	—	SCS
	do	do	do	—	—	June	1962	7.9	138,227.0	1.129	*60	.426	557	
4-17	Little Deer No. 1	Cattail Branch	Madonna, Md.	.74	.73	June	1958	—	6/23.37	—	—	—	—	SCS
	do	do	do	—	—	Nov.	1962	4.4	5/19.99	—	*60	6/1.055	1,378.7	
4-18	Mount Morris	Genesee River	Mount Morris, N. Y.	1,077	1,011	Nov.	1951	—	338,010	.347	—	—	—	CE
	do	do	do	—	—	May	1957	5.5	336,611	.345	7/—	.25	419	
	do	do	do	—	—	May	1963	5.9	335,393	.344	77	.20	335	
4-19	Patterson Creek #1	Patterson Creek	Endwell, N. Y.	4.3	4.3	Oct.	1968	—	891.40	.229	—	—	—	SCS
	do	do	do	—	—	Oct.	1970	2	887.95	.228	*80	6/.40	6/696	
4-20	Little Choconut #2B	West Branch Little Choconut	Choconut Center, N. Y.	1.64	1.64	Oct.	1968	—	272	.183	—	—	—	SCS
	do	do	do	—	—	Oct.	1970	2	271.38	.182	*80	6/.19	6/331	

POTOMAC, RAPPAHANNOCK, YORK, AND JAMES RIVER BASINS

5-1a	Lake Barcroft	Trib. of Potomac River	Falls Church, Va.	14.5	14.3	Jan.	1915	—	8/1,847	.142	—	—	—	SCS
	do	do	do	—	—	Feb.	1938	23.1	8/1,762	.134	*60	.257	336	
	do	do	do	—	—	Aug.	1957	19.5	9/2,092	.161	—	.728	950	
5-2	Pedlar	Pedlar River	Oronoco, Va.	33.21	33.07	Feb.	1907	—	1,860	—	—	—	—	SCS
	do	do	do	—	—	Feb.	1938	31	1,723	—	—	.134	—	
5-3	Burnt Mills	N. W. Br. Anacostia River	Silver Spring, Md.	27.0	26.97	May	1930	—	181	—	—	—	—	SCS
	do	do	do	—	—	Mar.	1938	7.8	95	—	*60	.408	533	
5-4b	Greenbelt Lake	Trib. of Indian Creek	Greenbelt, Md.	10/.82	.79	July	1936	—	196	*.312	—	—	—	SCS
	do	do	do	—	—	Feb.	1938	1.6	186	*.296	*60	7.91	10,337	
	do	do	do	—	—	Aug.	1957	19.5	151	*.240	*60	2.27	2,970	
	do	do	do	—	—	June	1968	10.8	11/147	.234	*60	1.52	1,945	
5-5a	Staunton	North River	Staunton, Va.	25	25	Dec.	1925	—	385	—	—	—	—	SCS
	do	do	do	—	—	Jan.	1940	14	373	—	—	.034	—	
	do	do	do	—	—	June	1957	17.5	350	—	—	.053	—	
5-6	Jackson	Occoquan Creek	Manassas, Va.	337	336.4	July	1930	—	4,500	—	—	—	—	SCS
	do	do	do	—	—	Aug.	1937	7.2	4,158	—	*60	.141	184	
5-7b	Triadelphia L. (Brighton D.)	Patuxent River	Brighton, Md.	81.4	80.0	Jan.	1942	—	12/20,222	.327	—	—	—	SCS
	do	do	do	—	—	Oct.	1950	8.3	20,089	.324	*50	.20	218	
	do	do	do	—	—	Sept.	1958	7.9	19,633	.317	*50	.72	784	
	do	do	do	—	—	Aug.	1964	5.9	19,045	.308	61.1	1.25	1,663	
5-8	Gordon Lake	Evitts Creek	Cumberland, Md.	64	13/4	Sept.	1913	—	3,129	—	—	—	—	SCS
	do	do	do	—	—	Apr.	1940	26.6	3,004	—	—	14/.090	—	
5-9	Thomas W. Koon Lake	do	do	60	59.6	Mar.	1932	—	7,312	—	—	—	—	SCS
	do	do	do	—	—	Apr.	1940	8.1	7,294	—	—	.036	—	
5-10	Savage River Dam	Savage River	Bloomington, Md.	105.0	104.44	Mar.	1952	—	20,300	.172	—	—	—	SCS
	do	do	do	—	—	Mar.	1956	4.0	20,020	.169	*60	.643	840	
5-11	Rocky Gorge	Patuxent River	Laurel, Md.	132.8	50.14	Mar.	1954	—	21,390	—	—	—	—	SCS
	do	do	do	—	—	Aug.	1964	10.4	20,789	—	67	1.15	1,678	
5-12	South River, Site 26	Inch Branch	Waynesboro, Va.	2.7	2.7	May	1956	—	610.4	.28	—	—	—	SCS
	do	do	do	—	—	Nov.	1970	14.5	607.0	.28	*60	.087	110	
5-13	Wilde Lake	Trib. Little Patuxent	Columbia, Md.	1.88	1.85	Sept.	1966	—	196.97	.140	—	—	—	SCS
	do	do	do	—	—	Aug.	1968	1.9	170.99	.122	*60	15/7.39	16/11,278	
	do	do	do	—	—	Aug.	1969	1.0	163.72	.117	*60	3.93	5,133	

CHOWAN, ROANOKE, TAR, NEUSE, AND CAPE FEAR RIVER BASINS

6-1	Lake Apex	Swift Creek	Apex, N. C.	4.0	4.0	—	1925	—	106	—	—	—	—	SCS
	do	do	do	—	—	June	1941	16	94	—	—	.19	—	
6-2	Franklinton	Sallie Keeney Creek	Franklinton, N. C.	1.13	1.12	Jan.	1925	—	34.7	—	—	—	—	SCS
	do	do	do	—	—	May	1938	13.3	27.3	—	67	.509	743	
6-3	Greensboro (L. Brandt)	Reddy Fork	Greensboro, N. C.	74.1	73.4	Feb.	1923	—	2,870	—	—	—	—	SCS
	do	do	do	—	—	Aug.	1934	11.5	2,610	—	*60	.308	402	
6-4	High Point	Deep River	High Point, N. C.	62.8	62.3	Jan.	1928	—	4,354	—	—	—	—	SCS
	do	do	do	—	—	Aug.	1934	6.5	4,135	—	50.6	.541	596	
	do	do	do	—	—	Apr.	1938	3.75	4,038	—	—	.416	658	

1/ Includes estimated 112 acre-feet passing through Shandaken Tunnel.
2/ Includes 103 acre-feet of sediment dredged in 1937-1939.
3/ Partial survey covering segments 1-14 in Stoney Brook Arm Only.
4/ Net sediment contributing area was 299.4 sq. mi. until 1933 when Prettyboy Dam was completed. This area was used in the 1943 calculations.
5/ Revised after 1961 survey.
6/ Conservation or sediment pool only.
7/ Not determined - assumed equal to that determined in 1963.
8/ Based on original spillway crest elevation 205 feet m. s. l.
9/ Based on spillway crest elevation 210 feet m. s. l. and estimated capacity of 2,180 acre-feet resulting from 1942 addition to top of dam.
10/ Revised 1968.
11/ 9 acre-feet gained by dredging.
12/ Revised due to movable control gates.
13/ Koon Lake, upstream, was built in 1932.
14/ Based on total sediment in both Gordon Lake and Koon Lake.
15/ Does not include 4.34 acre-feet dredged.
16/ Includes 4.34 acre-feet dredged in early spring 1968.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
CHOWAN, ROANOKE, TAR, NEUSE, AND CAPE FEAR RIVER BASINS (CONTINUED)													
6-5	Lake Michie	Flat River	Durham, N. C.	167.5	166.7	Apr. 1926	—	12,671	—	—	—	—	SCS
	do	do	do	—	—	Jan. 1935	8.75	12,276	—	—	0.271	—	
6-6	Sanford City	Lick Creek	Sanford, N. C.	3.75	3.73	Feb. 1927	—	103	—	—	—	—	SCS
	do	do	do	—	—	June 1941	14.33	95	—	—	.150	—	
6-7	University Lake	Morgan Creek	Chapel Hill, N. C.	30.6	30.3	June 1932	—	1,915	—	—	—	—	SCS
	do	do	do	—	—	Apr. 1935	2.9	1,851	—	—	.728	—	
6-8	Roxboro City Lake	Satterfield Creek	Roxboro, N. C.	7.62	7.52	Apr. 1924	—	531.2	—	—	—	—	SCS
	do	do	do	—	—	June 1941	17.2	460.3	—	—	.548	—	
	do	do	do	—	—	Nov. 1946	5.4	448.4	—	—	1/.342	—	
6-9	Burlington Municipal	Stony Creek	Burlington, N. C.	105.2	105.0	June 1928	—	1,488	—	—	—	—	SCS
	do	do	do	—	—	May 1938	10.0	1,325	—	63	.155	213	
	do	do	do	—	—	Sept. 1949	11.3	1,202	—	—	.104	143	
6-10a	Walnut Cove	Dan River	Walnut Cove, N. C.	397	397	— 1924	—	974	.002	—	—	—	CE
	do	do	do	—	—	Apr. 1932	2/9	209	.001	*43	.214	200	
6-11	John H. Kerr	Roanoke River	South Hill, Va.	7,800	7,391	July 1952	—	2,808,400	.494	—	—	—	CE
	do	do	do	—	—	Dec. 1959	7.4	2,750,349	.484	32.6	1.06	754	
6-12	Philpott	Smith River	Bassett, Va.	212	185.7	Dec. 1951	—	201,900	.966	—	—	—	CE
	do	do	do	—	—	Nov. 1960	8.9	198,000	.949	46.2	2.08	2,103	
PEE DEE, SANTEE, AND EDISTO RIVER BASINS													
7-1	Chester	Sandy River	Chester, S. C.	16.05	15.92	Nov. 1926	—	682	—	—	—	—	SCS
	do	do	do	—	—	Oct. 1937	11	585	—	—	.55	—	
7-2	Lancaster	Turkey Quarter Creek	Lancaster, S. C.	9.40	9.34	Feb. 1925	—	242	—	—	—	—	SCS
	do	do	do	—	—	June 1938	13.4	190	—	65.3	.417	593	
7-3a	Spartanburg Municipal	South Pacolet River	Fingerville, S. C.	91.33	90.8	May 1926	—	3/3,906	—	—	—	—	SCS
	do	do	do	—	—	July 1934	8.2	3/3,199	—	—	3/.412	—	
	do	do	do	—	—	Mar. 1947	12.7	3/2,891	—	—	3/.268	—	
	do	do	do	—	—	Oct. 1965	18.6	3/2,593	—	—	3/.176	—	
	do	do	do	—	—	Oct. 1965	18.6	3/3,109	—	—	—	—	
7-4	Appalachie	South Tyger River	Greer, S. C.	63.0	62.8	— 1904	—	2,900	—	—	—	—	SCS
	do	do	do	—	—	July 1934	30	[illegible]	—	—	.48	—	
7-5	Albemarle City Lake	Long Creek	Albemarle, N. C.	33.0	32.5	Feb. 1924	—	1,070	—	—	—	—	SCS
	do	do	do	—	—	Aug. 1939	15.5	918	—	—	.302	—	
7-6	Cannon Lake	Buffalo Creek	Kannapolis, N. C.	18.0	17.7	July 1939	—	2,600	—	—	—	—	SCS
	do	do	do	—	—	June 1941	1.9	2,574	—	—	.774	—	
7-7	Lake Concord	Chambers & Rose Branch	do	4.7	4.54	Mar. 1925	—	1,201	—	—	—	—	SCS
	do	do	do	—	—	May 1935	10.2	1,122	—	—	1.71	—	
7-8	Entwhistle No. 3	Hitchcock Creek	Roberdell, N. C.	168.0	5/18.0	— 1892	—	184	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1940	48.0	163	—	—	.024	—	
7-9	Bury	Little River	Troy, N. C.	269	269	— 1915	—	1,104	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1940	25.0	976	—	—	.019	—	
7-10	High Rock	Yadkin River	Salisbury, N. C.	3,930	3,863	Nov. 1927	—	289,432	—	—	—	—	SCS
	do	do	do	—	—	Aug. 1935	7.8	275,516	—	—	.462	—	
7-11	Lake Lee	Richardson Creek	Monroe, N. C.	50.50	50.34	Apr. 1927	—	821	—	—	—	—	SCS
	do	do	do	—	—	June 1938	11.1	652	—	61.8	.302	406	
7-12	Pee Dee Mfg. Co.	Hitchcock Creek	Rockingham, N. C.	176	5/25	— 1874	—	464	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1940	66	404	—	—	.036	—	
7-13	Salem	Salem Creek	Winston-Salem, N. C.	27.68	27.26	Nov. 1919	—	3,099	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1939	19.8	2,860	—	38.0	.444	367	
7-14	Norwood L. (Tillery)	Pee Dee River	Mt. Gilead, N. C.	4,600	6/431	July 1928	—	136,823	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1940	11.75	133,300	—	—	.696	—	
7-15	Lexington	Leonards Creek	Lexington, N. C.	6.75	6.66	Aug. 1935	—	462	—	—	—	—	SCS
	do	do	do	—	—	Apr. 1940	4.6	441	—	45.5	.692	685	
	do	do	do	—	—	May 1951	11.1	419	—	*45.5	.299	296	
7-16	Third Creek Site No. 7	Third Creek	Statesville, N. C.	4.84	4.74	Sept. 1957	—	961.4	.186	—	—	—	SCS
	do	do	do	—	—	Mar. 1962	4.5	948.5	.184	7/62.2	.61	826	
	do	do	do	—	—	Mar. 1965	3.0	945.9	.183	7/62.2	.18	244	
	do	do	do	—	—	Apr. 1969	4.1	943.1	.183	7/62.2	.14	190	

SAVANNAH, OGEECHEE, AND ALTAMAHA RIVER BASINS

8-1	Lake Issaqueena	Six Mile Creek	Clemson, S. C.	14.02	8/13.84	June	1938	—	1,836	—	—	—	—	SCS
	—do—	—do—	—do—	—	9/13.86	Apr.	1941	2.9	1,743	—	49.82	2.22	2,410	
	—do—	—do—	—do—	—	—	Oct.	1949	8.5	1,626	—	90.9	1.03	1,140	
8-2	Lloyd Shoals	Ocmulgee River	Jackson, Ga.	1,414	1,407	Dec.	1910	—	112,538	—	—	—	—	SCS
	—do—	—do—	—do—	—	—	Mar.	1935	24.3	98,578	—	*60	.408	532	
8-3a	North Fork Broad No. 2	Denman's Creek	Toccoa, Ga.	.96	.96	Jan.	1956	—	10/198	.182	—	—	—	SCS
	—do—	—do—	—do—	—	—	June	1959	3.4	196.2	.179	55.1	.563	676	
	—do—	—do—	—do—	—	—	Apr.	1970	10.8	192.8	.176	55.1	.335	402	
8-4a	North Fork Broad No. 6	Bear Creek	Mize, Ga.	3.62	3.90	July	1956	—	10/780.9	.199	—	—	—	SCS
	—do—	—do—	—do—	—	—	June	1959	2.92	770.6	.197	74.8	1.02	1,645	
	—do—	—do—	—do—	—	—	Apr.	1970	10.8	751.55	.192	74.8	.50	814	
8-5a	North Fork Broad No. 11	Tom's Creek	Eastanollee, Ga.	3.79	3.67	July	1956	—	10/792.4	.193	—	—	—	SCS
	—do—	—do—	—do—	—	—	June	1959	2.9	777.6	.190	73.8	1.39	2,234	
	—do—	—do—	—do—	—	—	Apr.	1970	10.9	763.9	.186	73.8	.34	547	
8-6	North Fork Broad No. 1	North Fork Broad River	Toccoa, Ga.	3.75	3.70	June	1958	—	633.2	.148	—	—	—	SCS
	—do—	—do—	—do—	—	—	Apr.	1970	11.9	624.16	.146	63.2	.205	282	
8-7	North Fork Broad No. 14	Tom's Creek	—do—	1.2	1.191	Oct.	1956	—	281.1	—	—	—	—	SCS
	—do—	—do—	—do—	—	—	Mar.	1962	7.4	276.8	—	11/68.6	.49	732	
	—do—	—do—	—do—	—	—	Nov.	1964	2.7	272.0	—	—	1.49	—	
	—do—	—do—	—do—	—	—	Apr.	1969	4.5	267.0	—	12/66.2	.93	1,341	

SATILLA, ST. MARYS, ST. JOHNS, AND SUWANNEE RIVER BASINS

9-

SOUTHERN FLORIDA DRAINAGE

10-

APALACHICOLA AND OCHLOCKONEE RIVER BASINS

11-1	Newnan	Bolton Mill Creek	Newnan, Ga.	1.39	1.34	June	1924	—	386		—	—	—	SCS
	—do—	—do—	—do—	—	—	Nov.	1937	13.4	358	—	*50	1.45	1,980	
	—do—	—do—	—do—	—	—	Feb.	1965	7.3	354	—	—	.41	446	
11-2	Sky Lake	Trib. of Chickamauga Creek	Helen, Ga.	2.34	2.31	June	1925	—	218.4	—	—	—	—	SCS
	—do—	—do—	—do—	—	—	May	1956	31	176.6	—	67	.58	846	
11-3	Sautee Creek Site #13	Bean Creek	Cleveland, Ga.	2.9	2.8	Dec.	1959	—	654.8	.116	—	—	—	SCS
	—do—	—do—	—do—	—	—	Nov.	1970	10.9	535.2	.095	75.8	3.92	6,472	

CHOCTAWHATCHEE, YELLOW, ESCAMBIA, AND ALABAMA RIVER BASINS

12-1	Sequoyah	Small Branches	Jasper, Ga.	1.60	1.51	July	1929	—	890	—	—	—	—	SCS
	—do—	—do—	—do—	—	—	July	1939	10.0	865	—	—	1.66	—	
12-2	White Manganese No. 6	Pettit Creek	Cartersville, Ga.	12.46	13/11.0	Oct.	1929	—	1,021	—	—	—	—	SCS
	—do—	—do—	—do—	—	—	Nov.	1938	9.2	900	—	63.7	1.30	1,460	
12-3	Lake Auburn	Town Creek (Trib. of)	Auburn, Ala.	1.6	1.6	Feb.	1931	—	102	—	—	—	—	SCS
	—do—	—do—	—do—	—	—	June	1937	6.3	95	—	—	.46	—	
12-4	Lay	Coosa River	Clanton, Ala.	9,087	9,076.5	Dec.	1913	—	156,525	.014	—	—	—	SCS
	—do—	—do—	—do—	—	—	May	1936	22.3	138,520	.012	—	.099	—	
12-5	Lake Purdy	Little Cahaba River	Birmingham, Ala.	41.74	40.22	Sept.	1910	—	19,080	—	—	—	—	SCS
	—do—	—do—	—do—	—	—	Nov.	1935	25.2	18,596	—	—	.479	—	
12-6	Carroll Lake	Curtis Creek	Carrollton, Ga.	7.45	7.18	Oct.	1948	—	1,448.0	.182	—	—	—	SCS
	—do—	—do—	—do—	—	—	May	1957	8.5	1,325.4	.167	42.5	2.0	1,851	
12-7	Temple Reservoir	Webster Branch	Temple, Ga.	.63	.61	June	1954	—	65.5	.097	—	—	—	SCS
	—do—	—do—	—do—	—	—	May	1957	3	60.5	.090	44.2	2.76	2,638	
12-8	High Pine Dam No. 5	Trib. of High Pine	Roanoke, Ala.	1.8	1.55	Mar.	1961	—	389.7	*.163	—	—	—	
	—do—	—do—	—do—	—	—	May	1970	9.17	385.3	*.161	14/53.99	.32	376.3	SCS

TOMBIGBEE, PASCAGOULA, AND PEARL RIVER BASINS

13-1	Bayview	Village Creek	Birmingham, Ala.	72.3	71.6	May	1911	—	11,866	—	—	—	—	SCS
	—do—	—do—	—do—	—	—	Dec.	1935	24.6	9,514	—	—	1.34	—	
13-2a	Lake Harris	Yellow Creek	Tuscaloosa, Ala.	30.0	29.8	Feb.	1929	—	2,421	.081	—	—	—	SCS
	—do—	—do—	—do—	—	—	Nov.	1935	6.75	2,373	.079	—	.239	—	
	—do—	—do—	—do—	—	—	Aug.	1953	17.75	13/2,636	.066	59.9	.110	144	
13-3	N. C. Crain Farm Pond #2	Bogue Lusa Creek	Bogalusa, La.	.13	.13	Aug.	1949	—	7.631	.064	—	—	—	SCS
	—do—	—do—	—do—	—	—	May	1963	13.5	6.963	.060	—	.377	—	

LOWER MISSISSIPPI RIVER BASIN (BATCHEZ TO THE MOUTH)
Calcasieu, Mermentau, and Vermilion River Basins

14-

1/ Excluding 2.04 acre-feet of sediment dredged from lake in March 1962.

2/ Assume storage depletion occurred in 9 years, see H. D. 65, 75th Congress, 1st. session ("308" report).

3/ Elevation of dam raised from 775.0 to 777.1 feet Sept. 1956. Data based on original elevation of 775.0 feet and original capacity of 3506 ac.-ft.

4/ Present storage capacity at elevation 777.1 feet.

5/ Lofbotters' dam upstream

6/ Narrows upstream

7/ Average of 39 samples collected in 1962.

8/ In 1960

9/ In 1969

10/ Changed on basis of 1970 survey.

11/ Average of 9 samples.

12/ Average of 10 samples.

13/ Drainage area above reservoir No. 3 excluded.

14/ Weighted average: Submerged sediment 51.75 pcf- aerated assumed 83 pcf.

15/ With 5-foot flashboards added in 1967.

* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

LOWER MISSISSIPPI RIVER BASINS (HELENA TO NATCHEZ)
Yazoo, Big Black, and Ouachita River Basins

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE -FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
15-1	Lake Hamilton	Ouachita River	Hot Springs, Ark.	1,421	1,413.4	Dec. 1930	—	156,345	—	—	—	—	SCS
	do	do	do	—	—	May 1950	19.4	153,279	—	*50	0.11	120	
15-2	Lake Winona	Alum Fork Saline River	Little Rock, Ark.	43.0	41.3	Oct. 1937	—	34,533	—	—	—	—	SCS
	do	do	do	—	—	May 1950	12.6	34,340	—	*50	.37	403	
15-3	O. P. White Pond	Trib. of Chewalla Creek	Holly Springs, Miss.	.0097	.0089	Oct. 1947	—	1.46	—	—	—	—	SCS
	do	do	do	—	—	Jan. 1951	3.3	1.07	—	63.65	13.1	18,200	
15-4	B. H. Honnecucker Pond	Trib. of Coldwater River	Slayden, Miss.	.0625	.0597	Jan. 1947	—	12.0	—	—	—	—	SCS
	do	do	do	—	—	Jan. 1951	4.0	8.35	—	60.09	15.3	20,000	
15-5	C. S. Hurdle Pond	do	do	.244	.234	Dec. 1946	—	36.4	—	—	—	—	SCS
	do	do	do	—	—	Jan. 1951	4.1	29.0	—	62.9	7.78	10,700	
15-6	Agnes Jones Pond	Trib. of Coldwater River	Holly Springs, Miss.	.0478	.0451	Oct. 1946	—	8.80	—	—	—	—	SCS
	do	do	do	—	—	Jan. 1951	4.3	6.40	—	67.27	12.4	18,200	
15-7	Lee Johnson Pond	Trib. of Red Banks Creek	do	.0833	.0784	Sept. 1948	—	17.8	—	—	—	—	SCS
	do	do	do	—	—	Jan. 1951	2.3	14.8	—	76.13	16.6	27,500	
15-8	P. T. McAlexander	Trib. of Coldwater River	do	.0139	.0128	Aug. 1942	—	2.46	—	—	—	—	SCS
	do	do	do	—	—	Jan. 1951	8.5	1.11	—	84.24	12.4	22,800	
15-9	Lake Shahinka	Trib. of Camp Creek	Olive Branch, Miss.	.4153	.3663	Jan. 1934	—	306	—	—	—	—	SCS
	do	do	do	—	—	Feb. 1951	17.0	256	—	61.65	8.03	10,800	
15-10	Lake Woodland	do	do	.5045	.4797	Feb. 1945	—	96	—	—	—	—	SCS
	do	do	do	—	—	Feb. 1951	6.0	78.1	—	39.9	5.52	4,800	
15-11	C. L. Patton Pond	Trib. of Red Banks Creek	Warsaw, Miss.	.1019	.0954	Feb. 1942	—	15.9	—	—	—	—	SCS
	do	do	do	—	—	Feb. 1951	9.0	12.1	—	66.8	4.48	6,520	
15-12	G. B. Langston, Jr., Pond	Trib. of Red Banks Creek	Warsaw, Miss.	.0156	.0116	Feb. 1916	—	12.3	—	—	—	—	SCS
	do	do	do	—	—	Feb. 1951	35.0	9.66	—	63.3	6.55	9,040	
15-13	Fletcher Hurdle Pond (North)	Trib. of Byhalia Creek	Victoria, Miss.	.0406	.0389	Feb. 1947	—	4.61	—	—	—	—	SCS
	do	do	do	—	—	Feb. 1951	4.0	2.67	—	63.0	12.5	17,100	
15-14	Fletcher Hurdle Pond (South)	do	do	.0297	.0279	Feb. 1947	—	5.48	—	—	—	—	SCS
	do	do	do	—	—	Feb. 1951	4.0	3.61	—	83.1	16.8	30,400	
15-15	C. C. Stevenson Pond	Trib. of Cuffawa Creek	Holly Springs, Miss.	.0342	.0320	Aug. 1948	—	6.47	—	—	—	—	SCS
	do	do	do	—	—	Feb. 1951	2.7	4.23	—	78.8	26.0	44,600	
15-16	Gayoso Lake	Trib. of Mississippi River	Horn Lake, Miss.	.2196	.2047	Feb. 1942	—	58.7	—	—	—	—	SCS
	do	do	do	—	—	Feb. 1951	9.0	53.0	—	51.85	3.13	3,530	
15-17	Ben O. Pettis Pond	Trib. of Toby Tubby Creek	Oxford, Miss.	.0075	.0067	Feb. 1916	—	1.63	—	—	—	—	SCS
	do	do	do	—	—	Feb. 1951	35.0	1.07	—	58.72	2.39	3,060	
15-18	C. D. Williams Pond	Trib. of Hudson Creek	do	.0450	.0418	Mar. 1928	—	9.93	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1951	23.0	6.70	—	55.72	3.35	4,070	
15-19	R. X. Williams	Trib. of Yocona River	Taylor, Miss.	.0239	.0210	Sept. 1946	—	4.48	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1951	4.5	2.54	—	47.74	20.5	21,300	
15-20	Henry W. Ramsey Pond	Trib. of Sarter Creek	Oxford, Miss.	.1375	.1289	Jan. 1933	—	31.0	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1951	18.2	24.9	—	41.75	2.61	2,370	
15-21	Dr. Bramlett Pond	Trib. of Pumpkin Creek	do	.8672	.8131	Jan. 1937	—	227	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1951	14.2	214	—	*37	1.12	913	
15-22	A. S. Kyle Pond	Trib. of Tallahatchie R.	Batesville, Miss.	.0456	.0398	Mar. 1945	—	17.4	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1951	6.0	16.6	—	52.06	3.14	3,560	
15-23	Ben P. Smith Pond	Trib. of Pigeon Roost Cr.	Holly Springs, Miss.	.0278	.0264	July 1947	—	3.77	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1951	3.7	3.29	—	67.58	4.89	7,200	
15-24	A. L. Rodman Pond	Trib. of Arkabutla Creek	Arkabutla, Miss.	.1563	.1458	Sept. 1945	—	30.5	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1951	5.5	29.0	—	*90	1.95	2,120	
15-25	Charles Dockery Pond	Trib. of Hurricane Creek	Eudora, Miss.	.0644	.0566	Jan. 1946	—	23.0	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1951	5.2	22.0	—	*44	3.60	3,420	
15-26b	Arkabutla Reservoir	Coldwater River	Arkabutla, Miss.	1,000	948	Apr. 1939	—	525,300	.499	—	—	—	CE
	do	do	do	—	—	Dec. 1947	6.33	521,540	.495	*60	.627	819	
	do	do	do	—	—	May 1962	14.42	513,060	.487	*60	.620	810	
15-27a	W. W. Murphy Pond	Trib. of Jones Creek	Holly Springs, Miss.	.208	.200	Nov. 1953	—	1/22.48	*.194	—	—	—	ARS
	do	do	do	—	—	May 1956	2.5	19.72	*.169	*90	2/8.4	2/16,465.7	
	do	do	do	—	—	Sept. 1959	3.3	17.84	*.152	*90	2/3.8	2/7,448.8	
	do	do	do	—	—	Nov. 1963	4.2	15.77	*.135	*90	2/3.5	2/6,860.7	
15-28a	Powerline Dam	East Goose Creek	Oxford, Miss.	.486	.477	Apr. 1953	—	1/30.54	—	—	—	—	ARS
	do	do	do	—	—	May 1955	2.1	26.26	—	66	2/5.66	2/8,136.1	
	do	do	do	—	—	Oct. 1958	3.4	21.34	—	70	2/5.87	2/8,949.4	
	do	do	do	—	—	June 1960	1.66	20.07	—	70	2/5.30	2/8,080.4	
	do	do	do	—	—	Aug. 1962	2.15	19.40	—	*70	2/1.38	2/2,103.9	

15-29a	Andrew Smith Pond	Trib. of Jones Creek	Holly Springs, Miss.	.327	.300	Nov.	1953	—	1/22.72	*.123	—	—	—	ARS
	do	do	do	—	—	May	1956	2.5	18.83	*.102	*90	2/6.2	2/12,153.2	
	do	do	do	—	—	Sept.	1959	3.3	17.77	*.096	*90	2/2.07	2/4,057.6	
	do	do	do	—	—	Nov.	1963	4.1	16.47	*.089	*90	2/1.87	2/3,665.6	
15-30a	Sardis Reservoir	Little Tallahatchie River	Sardis, Miss.	1,545	1,456	Mar.	1937	—	1,569,900	.934	—	—	—	CE
	do	do	do	—	—	May	1960	20.6	1,549,336	.922	*60	.687	898	
15-31	Enid Reservoir	Yocona River	Enid, Miss.	560	516	—	1940	—	660,030	1.073	—	—	—	CE
	do	do	do	—	—	—	1951 3/	—	—	—	—	—	—	
	do	do	do	—	—	May	1961	9.83	657,201	1.068	*60	.558	729	
15-32	Grenada Reservoir	Yalobusha River	Grenada, Miss.	1,320	1,219	—	1942	—	1,337,400	.969	—	—	—	CE
	do	do	do	—	—	July	1953 4/	—	—	—	—	—	—	
	do	do	do	—	—	May	1965	11.43	1,320,020	.957	*60	1.205	1,575	

LOWER MISSISSIPPI RIVER BASIN (CHESTER TO HELENA)
St. Francis River Basin

16-1	Grisham	Lost Creek	Bismark, Mo.	.46	.45	Oct.	1930	—	24.05	*.077	—	—	—	SCS
	do	do	do	—	—	July	1939	8.8	19.56	*.063	75.4	1.133	1,860	
16-2	Mountain Lake	Trib. of Rings Creek	Patterson, Mo.	1.90	1.87	—	1927	—	87.7	—	—	—	—	SCS
	do	do	do	—	—	July	1939	12	82.9	—	54.8	.213	254	
16-3	Shepherd Mountain	Trib. of Stouts Creek	Ironton, Mo.	3.99	3.96	—	1929	—	171	—	—	—	—	SCS
	do	do	do	—	—	July	1939	10	158	—	64	.338	471	
16-4	Loch Mary	Brown Creek	Earlington, Ky.	3.81	3.65	—	1888	—	1,228	—	—	—	—	SCS
	do	do	do	—	—	Dec.	1908	20	1,184	—	*60	.600	784	
16-5	Carbondale	Piles Fork	Carbondale, Ill.	3.00	2.77	Aug.	1926	—	1,386	—	—	—	—	IWS
	do	do	do	—	—	Sept.	1948	22.1	1,193	—	73.9	3.15	5,070	
16-6	Dering Coal Co. Pond	Trib. of Wolf Creek	Eldorado, Ill.	.219	.206	—	1919	—	89.3	—	—	—	—	IWS
	do	do	do	—	—	Oct.	1949	30	73.0	—	*76	2.64	4,370	
16-7	Eldorado	Wolf Creek	do	2.23	1.87	Oct.	1920	—	844.4	—	—	—	—	IWS
	do	do	do	—	—	Oct.	1949	29	726.0	—	*67	2.18	3,180	
16-8	West Frankfort	Tilley Creek	West Frankfort, Ill.	4.03	3.75	Aug.	1926	—	5/1,608.4	—	—	—	—	IWS
	do	do	do	—	—	Sept.	1936	10.1	1,515.0	—	—	2.46	—	
	do	do	do	—	—	July	1949	12.8	1,487.8	—	—	6/.933	—	
16-9	Pineview (Lower)	South Fork Jonses Creek	Farmington, Mo.	.63	.07	—	1932	—	8.9	—	—	—	—	SCS
	do	do	do	—	—	—	1939	7	8.2	—	*60	1.43	1,870	
16-10	Pineview (Middle)	do	do	.56	.06	—	1930	—	30.9	—	—	—	—	SCS
	do	do	do	—	—	—	1939	9	29.1	—	*60	3.3	4,310	
16-11	Pineview (Upper)	do	do	.49	.48	—	1928	—	8.2	*.025	—	—	—	SCS
	do	do	do	—	—	—	1938 7/	10	5.4	*.016	*65	.583	825	
16-12	Killarney	Big Creek	Annapolis, Mo.	51	51	—	1910	—	818	—	—	—	—	SCS
	do	do	do	—	—	—	1939	29	622	—	*60	.133	174	
16-13a	Wappapello	St. Francis River	Poplar Bluff, Mo.	1,310	1,206	July	1940 8/	—	625,000	.540	—	—	—	CE
	do	do	do	—	—	July	1947	7.0	624,651	.539	—	9/.0414	—	
	do	do	do	—	—	Mar.	1964	16.7	613,161	.530	—	.5705	—	
16-14a	Lake Miller	Casey Fork	Mt. Vernon, Ill.	4.65	4.43	Feb.	1944	—	1,746	.613	—	—	—	IWS
	do	do	do	—	—	June	1960	16.3	1,659	.582	34.0	1.20	888	
16-15	Flucks Lake	Unnamed	Marion, Ill.	.339	.316	—	1919	—	58.1	.265	—	—	—	IWS
	do	do	do	—	—	—	1951	32	46.8	.214	56.6	1.11	1,368	
16-16	Baker's Lake	do	do	.26	.25	—	1937	—	24.0	.143	—	—	—	IWS
	do	do	do	—	—	Aug.	1951	14	21.7	.129	36.8	.64	512	
16-17	Crab Orchard Lake	Crab Orchard Creek	Carbondale, Ill.	171	160	May	1940	—	67,320	.611	—	—	—	IWS
	do	do	do	—	—	July	1951	11.2	63,894	.580	47.5	1.91	1,976	
16-18	Marion	Limb Branch	Marion, Ill.	6.5	6.31	—	1921	—	705	.168	—	—	—	IWS
	do	do	do	—	—	July	1951	30	590	.141	34.5	.61	458	
16-19	Little Grassy Lake	Little Grassy	Carbondale, Ill.	15.7	14.2	Mar.	1942	—	25,741	2.543	—	—	—	IWS
	do	do	do	—	—	July	1951	9.3	25,365	2.906	38.7	2.85	2,402	
16-20	Herrin Reservoir No. 1	Unnamed	Herrin, Ill.	1.78	1.70	—	1913	—	199	.173	—	—	—	IWS
	do	do	do	—	—	—	1951	38	178	.155	27.5	.32	192	
16-21	Knights of Pythias Lake	do	Marion, Ill.	.33	.32	Dec.	1925	—	74.6	.350	—	—	—	IWS
	do	do	do	—	—	Aug.	1951	25.7	64.7	.304	62.9	1.22	1,671	
16-22a	Lake Ashley	Trib. of Muddy River	Ashley, Ill.	1.24	1.22	—	1941	—	150	.201	—	—	—	IWS
	do	do	do	—	—	Aug.	1954	14	138	.184	44.5	.69	692	
16-23	Christopher	Trib. of King Creek	Christopher, Ill.	.925	.858	—	1925	—	383.94	.598	—	—	—	IWS
	do	do	do	—	—	July	1960	35	353.59	.551	37.1	1.01	816.1	

1/ Sediment or conservation pool only.
2/ Includes sediment within original flow line of reservoir and above conservation pool.
3/ Original sediment range surveys.
4/ Used as beginning date of sediment deposits.
5/ Spillway crest raised from 439 to 441.77 ft. m.s.l. in April 1943. All data computed on basis of 441.77 ft. at spillway elevation.
6/ Net sediment volume in 1949 was 120.5 ac.-ft. due to compaction of earlier deposits.
7/ Dam failed spring 1938, survey conducted July 24, 1939.
8/ Original data from topographic survey of 1935-36.
9/ Based on incomplete resurvey; 1963 value of 0.57 is more reliable.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
			LOWER MISSISSIPPI RIVER BASIN (CHESTER TO HELENA) St. Francis River Basin (Continued)										
16-24	Lake DuQuoin	Reese Creek	DuQuoin, Ill.	10.73	10.35	— 1937	—	2,003	.316	—	—	—	IWS
	—do—	—do—	—do—	—	—	July 1957	20	1,870	.295	38.3	0.66	534	
16-25	W. F. Farrell	Trib. of Harper Creek	Mt. Vernon, Ill.	.519	.512	— 1945	—	35.56	.103	—	—	—	IWS
	—do—	—do—	—do—	—	—	June 1960	15	27.41	.079	51.5	1.06	1,189	
16-26a	ICRR at Thompsonville	Trib. of Ewing Creek	Thompsonville, Ill.	1.799	1.725	— 1926	—	352.46	.283	—	—	—	IWS
	—do—	—do—	—do—	—	—	Aug. 1960	34	300.63	.241	48.7	.880	934	
16-27	Lake Johnston City	Lake Creek	Johnston City, Ill.	3.85	3.75	— 1922	—	471	.195	—	—	—	IWS
	—do—	—do—	—do—	—	—	Aug. 1957	35	394	.163	48.3	.59	620	
16-28	Valier Outing Club	Andy Creek	Valier, Ill.	2.47	2.37	— 1922	—	369	.208	—	—	—	IWS
	—do—	—do—	—do—	—	—	Aug. 1957	35	320	.180	41.17	.59	530	
16-29	West Frankfort (New)	Stevens Creek	West Frankfort, Ill.	7.622	7.288	— 1945	—	2,656.7	.502	—	—	—	IWS
	—do—	—do—	—do—	—	—	July 1960	15	2,390.9	.453	35.9	2.41	1,884.4	
16-30	Lake Reel-Em-In (Walker Lake)	Unnamed	Dixon, Ky.	2.05	1.96	— 1948	—	659.0	.378	—	—	—	SCS
	—do—	—do—	—do—	—	—	May 1964	16	515.4	.296	*65	4.6	6,512.2	
16-31	Rough River FRS No. 1	Buffalo Creek	Kingswood, Ky.	2.86	2.85	Nov. 1955	—	617	.238	—	—	—	SCS
	—do—	—do—	—do—	—	—	June 1970	14.66	596.12	.230	56.13	1/.50	1/611	
16-32	Obion Creek FRS #24	Unnamed Trib. Little Creek	Fancy Farm, Ky.	.37	.36	Nov. 1960	—	1/30.35	*.23	—	—	—	SCS
	—do—	—do—	—do—	—	—	Nov. 1965	5	1/28.80	*.22	*80	1/.86	1/1,498	
16-33	Mud River FRS No. 14	Antioch Creek	Sharon Grove, Ky.	8.59	8.56	Jan. 1964	—	1,379.67	.15	—	—	—	SCS
	—do—	—do—	—do—	—	—	May 1970	6.33	1,355.49	.15	73.70	1/.45	1/762	
16-34	Caney Creek WPS #8	Bennett Fork	Caneyville, Ky.	5.77	5.65	Nov. 1965	—	—	—	—	—	—	SCS
	—do—	—do—	—do—	—	—	May 1970	4.5	1,355.4	.33	77.75	1/1.01	1/1,710	
16-35	Big Reedy FRS #3	West Fork	Reedyville, Ky.	6.76	6.69	June 1966	—	998	.156	—	—	—	SCS
	—do—	—do—	—do—	—	—	Apr. 1970	5.8	976.95	.150	77.98	1/.54	1/912.4	
			OHIO RIVER BASIN (MADISON TO UNIONTOWN) Wabash River Basin										
17-1	Huntingburg (Upper)	Trib. of Patoka River	Huntingburg, Ind.	.67	.63	July 1914	—	137	*.264	—	—	—	SCS
	—do—	—do—	—do—	—	—	Oct. 1960	46.3	119	*.229	*60	.617	538	
17-2a	Oakland City #2	S. Fork Patoka River	Oakland City, Ind.	.52	.40	— 1941	—	2/850.8	*2.116	—	—	—	SCS
	—do—	—do—	—do—	—	—	Sept. 1960	19.0	2/835.8	*2.079	*60	1.937	1,720	
	—do—	—do—	—do—	—	—	Aug. 1965	24.9	811.0	*2.017	70	2.490	3,794	
17-3	Shafer Lake	Tippecanoe River	Monticello, Ind.	1,700	1,698	June 1923	—	14,722	*.016	—	—	—	SCS
	—do—	—do—	—do—	—	—	Aug. 1940	17.2	14,061	*.016	*75	.003	38	
17-4	Spring Mill	Mill Creek	Mitchell, Ind.	15.03	3/5.29	Oct. 1938	—	170	—	—	—	—	SCS
	—do—	—do—	—do—	—	—	Sept. 1948	9.9	127	—	*67	.975	1,420	
17-5	Greendale Lake	Connor's Branch	Xenia, Ill.	25.1	25.0	Aug. 1927	—	306	*.033	—	—	—	SCS
	—do—	—do—	—do—	—	—	Sept. 1940	13.1	260	*.030	*70	.140	213	
17-6	Ridge Lake	Trib. of Embarrass River	Charleston, Ill.	1.41	1.38	Apr. 1941	—	187.4	—	—	—	—	IWS
	—do—	—do—	—do—	—	—	Sept. 1947	6.4	171.9	—	72.4	1.75	2,760	
17-7	Vermilion Lake	N. Fork Vermilion River	Danville, Ill.	267	266	June 1915	—	4/8,443	*.061	—	—	—	SCS
	—do—	—do—	—do—	—	—	Oct. 1940	25.3	7,438	*.052	*70	.179	273	
17-8	Brown Park Lake	Trib. of Raccoon Creek	Flora, Ill.	1.34	1.33	— 1938	—	49.08	.097	—	—	—	IWS
	—do—	—do—	—do—	—	—	June 1959	21	37.75	.066	58.6	.40	510.5	
17-9	Craig & Davidson's Lake	—do—	Martinsville, Ill.	.465	.437	— 1947	—	187.75	.631	—	—	—	IWS
	—do—	—do—	—do—	—	—	June 1959	12	175.18	.588	33.14	2.46	1,775.6	
17-10	Farina Lake	None	Laclede, Ill.	.538	.531	— 1928	—	16.4	.097	—	—	—	IWS
	—do—	—do—	—do—	—	—	Aug. 1958	30	13.3	.066	61.96	.19	256.35	
17-11	Graham Lake	Trib. of Veale Creek	Washington, Ind.	.34	.31	Aug. 1950	—	106.8	—	—	—	—	SCS
	—do—	—do—	—do—	—	—	Aug. 1956	6	106.4	—	—	.218	—	
17-12a	ICRR at Bluford	Fourmile Creek	Bluford, Ill.	3.353	3.199	— 1926	—	670.7	.300	—	—	—	IWS
	—do—	—do—	—do—	—	—	June 1960	34	609.70	.273	43.5	.565	493	
17-13	Jones Pond	Trib. of Flat Creek	Otwell, Ind.	.034	.031	Oct. 1954	—	10.7	—	—	—	—	SCS
	—do—	—do—	—do—	—	—	Apr. 1956	1.5	10.6	—	—	2.10	—	
17-14a	Olney Reservoir (New)	Trib. of East Fork	Olney, Ill.	3.36	3.15	Sept. 1954	—	1,555.3	.723	—	—	—	IWS
	—do—	—do—	—do—	—	—	Aug. 1960	6.9	1,527.4	.708	30.3	1.70	1,460	
17-15	Patterson Lake	Trib. of Dismal Creek	Edgewood, Ill.	.959	.912	— 1926	—	316.75	.516	—	—	—	IWS
	—do—	—do—	—do—	—	—	July 1959	33	281.09	.458	48.56	1.18	1,247.5	
17-16a	Plum Creek No. 15	Little Plum Creek	Taylorsville, Ky.	1.03	1.02	Sept. 1956	—	224.3	.268	—	—	—	SCS
	—do—	—do—	—do—	—	—	Apr. 1959	2.5	219.6	.262	65	1.86	1,803	
	—do—	—do—	—do—	—	—	Oct. 1960	1.5	213.2	.256	53	4.19	4,837	
17-17	Plum Creek No. 17	Trib. of Little Plum Creek	—do—	.56	.55	Dec. 1956	—	126.3	.263	—	—	—	SCS
	—do—	—do—	—do—	—	—	Apr. 1959	2.33	123.7	.258	56	2.00	2,439	
	—do—	—do—	—do—	—	—	Oct. 1960	1.5	121.2	.252	47	3.11	3,180	
17-18	Steiner Lake	Trib. of Pond Creek	Fairfield, Ill.	.308	.295	— 1945	—	53.68	.261	—	—	—	IWS
	—do—	—do—	—do—	—	—	June 1960	15	48.62	.237	42.3	1.15	1,034.3	

17-19	Stevenson's Lake	Trib. of Kettering Branch	Martinsville, Ill.	.342	.327	—	1950	—	52.11	.238	—	—	—	IHB
	do	do	do	—	—	June	1959	9	46.5	.212	48.0	1.09	1,975.9	
17-20	Vevay Park Lake	Trib. of Range Creek	Greenup, Ill.	.294	.278	—	1906	—	67.41	.391	—	—	--	IHB
	do	do	do	—	—	June	1959	53	54.50	.316	47.62	.87	908.1	
17-21	Beaver Lake	Beaver Creek	Dubois, Ind.	3.95	3.57	Oct.	1955	—	2,486.0	*.787	—	—	—	SCS
	do	do	do	—	—	Oct.	1964	9	2,409.8	*.763	55	2.38	2,051	
17-22	Scottsburg Lake	Trib. of Muscatatuck	Scottsburg, Ind.	2.98	2.98	Fall	1949	—	769.02	*.296	—	—	--	SCS
	do	do	do	--	--	Oct.	1961	12	713.24	*.280	60.8	1.00	1,324.2	
17-23	Cagles Mill (Cataract Lake)	Mill Creek	Cloverdale, Ind.	295	287	Dec.	1952	—	5/232,370	1.123	—	—	--	CE
	do	do	do	—	—	Feb.	1962	9.17	230,906	1.116	62.8	.54	739	

TENNESSEE RIVER BASIN (BELOW HALES BAR DAM)
Cumberland and Green River Basins

18-1	Radnor Lake	Other Creek	Nashville, Tenn.	2.1	2.0	Mar.	1915	—	1,313	--	--	—	—	SCS
	do	do	do	—	—	Nov.	1940	25.7	1,279	—	—	.660	—	
18-2a	Lake Tandy	Little River	Hopkinsville, Ky.	6.10	6.00	—	1907	—	769	.135	—	—	--	SCS
	do	do	do	—	—	Jan.	1941	34	564	.099	43	1.00	936	
	do	do	do	--	—	June	1959	18.5	509	.089	43	.495	463	
18-3c	Great Falls	Caney Fork River	Rock Island, Tenn.	1,675	1,671	Sept.	1935	—	54,925	.026	—	—	--	TVA
	do	do	do	--	—	Aug.	1947	12.0	52,370	.023	*60	.127	166	
	do	do	do	--	--	Jan.	1954	6.4	51,836	.023	*60	.050	65	
	do	do	do	—	—	Sept.	1960	6.6	51,591	.022	*60	.022	29	
	do	do	do	—	—	Nov.	1969	9.2	51,281	.023	*60	.02	26	
18-4c	Guntersville	Tennessee River	Guntersville, Ala.	24,450	2,550	Nov.	1940	—	1,097,380	—	—	—	—	TVA
	do	do	do	—	—	June	1947	6.6	1,080,897	—	*55	.979	1,173	
	do	do	do	—	—	May	1956	8.9	1,073,164	—	*55	.341	408	
	do	do	do	—	--	July	1961	5.1	1,064,228	—	*55	.687	823	
18-5c	Wheeler	do	Town Creek, Ala.	29,590	6/5,033	Oct.	1936	—	7/1,122,000	—	—	—	--	TVA
	do	do	do	—	—	June	1947	10.7	7/1,071,717	—	*55	8/—	8/—	
	do	do	do	—	—	May	1953	6.0	8/1,061,411	—	*55	.341	408	
	do	do	do	—	—	June	1956	3.0	1,061,005	—	*55	.027	32	
	do	do	do	—	--	Aug.	1961	5.1	1,050,303	—	*55	.417	500	
18-6c	Wilson	do	Florence, Ala.	30,750	1,135	Dec.	1927	—	687,000	—	—	—	--	TVA
	do	do	do	—	—	Dec.	1931	3.0	674,000	—	*53	9/.479	552.9	
	do	do	do	—	—	Dec.	1936	5.0	652,000	—	*53	9/.490	565.6	
	do	do	do	—	—	Sept.	1946	9.8	651,000	—	*53	10/.120	138.5	
	do	do	do	—	—	June	1951	4.7	650,000	—	*53	10/.120	138.5	
	do	do	do	—	—	June	1956	5.0	648,000	—	*53	10/.401	462.0	
	do	do	do	--	—	Aug.	1961	5.1	641,000	—	*53	10/1.276	1,471.0	
18-7c	Pickwick Landing	do	Pickwick, Tenn.	32,820	1,997	Feb.	1938	—	1,130,313	—	—	—	--	TVA
	do	do	do	--	—	Sept.	1946	8.6	1,116,309	—	*53	.621	936	
	do	do	do	--	—	June	1951	4.7	1,118,082	—	*53	11/-.180	—	
	do	do	do	—	—	June	1956	5.1	1,116,811	—	*53	.125	144	
	do	do	do	—	—	Aug.	1961	5.2	1,105,256	—	*53	1.113	1,285	
18-8c	Kentucky	do	Gilbertsville, Ky.	40,200	7,131	Aug.	1944	—	7/2,879,549	—	—	—	--	TVA
	do	do	do	—	—	May	1951	4.7	7/2,855,440	—	*53	.719	830	
	do	do	do	—	—	July	1956	5.2	7/2,814,388	—	*53	1.107	1,278	
	do	do	do	—	—	Oct.	1961	5.2	7/2,790,855	—	*53	.635	733	
18-9	Dale Hollow	Obey River	Celina, Tenn.	935	887	Apr.	1943	—	1,706,000	1.533	—	—	—	CE
	do	do	do	—	—	June	1960	17.2	—	—	—	—	—	
18-10	Old Hickory	Cumberland River	Old Hickory, Tenn.	12/11,674	2,741	June	1954	—	467,000	--	—	—	—	CE
	do	do	do	—	—	June	1965	11	453,703	—	—	.441	—	
18-11	Wolf Creek (Lake Cumberland)	do	Jamestown, Ky.	5,789	5,690	Aug.	1950	—	6,089,000	.912	—	—	—	CE
	do	do	do	—	—	June	1963	12.83	—	—	—	—	—	
18-12	Rough River Reservoir	Rough River	Falls of Rough, Ky.	454	437.56	Dec.	1959	—	332,940	.866	—	—	--	CE
	do	do	do	—	—	July	1969	9.75	329,670	.857	70.27	.765	1,170	
18-13	Upper Green River #5 (Pilot)	South Fork	Stanford, Ky.	1.44	1.41	Sept.	1955	—	320.71	.23	—	—	--	SCS
	do	do	do	—	—	Apr.	1966	10.5	316.99	.18	59.9	1/.26	1/339	

OHIO RIVER BASIN (POINT PLEASANT TO MADISON)
Kanawha, Big Sandy, Licking, Kentucky, Scioto, and Miami River Basins

19-1	Lake Placid	Unnamed	Lancaster, Ky.	.20	.20	Oct.	1902	—	114	—	—	--	—	SCS
	do	do	do	—	—	Sept.	1961	58.9	95	—	—	13/2.50	—	
19-2	Radford	Little River	Radford, Va.	329	329	Aug.	1934	—	1,646	—	--	—	—	SCS
	do	do	do	—	—	July	1964	30	1,018	—	*70	.191	291	
19-3	Englewood	Stillwater	Dayton, Ohio	651	639	—	1927 14/	—	312,000	*.810	—	—	—	SCS
	do	do	do	—	—	—	1962	35	311,648	*.809	77.5	.037	63	
19-4	Germantown	Twin Creek	Germantown, Ohio	270	264	—	1927 14/	—	106,000	*.554	—	—	—	SCS
	do	do	do	—	—	—	1962	35	105,618	*.552	79.5	.097	168	
19-5b	Griggs	Scioto River	Columbus, Ohio	1,053	1,052	—	1905	—	4,563	.008	—	—	—	[illegible]
	do	do	do	—	—	—	1935	30	3,920	.007	*65	.020	28.3	
	do	do	do	—	—	—	1951	16	3,928	.007	59.4	—	—	
	do	do	do	—	—	—	1964	13	3,737	.007	—	.024	—	

1/ Sediment pool only.
2/ Items are based on corrected instrument surveys of shore line and range locations as used in the 1965 survey.
3/ The noncontributing drainage area is chiefly closed or plugged limestone sinkholes.
4/ In July 1925. In June 1915, spillway elevation was 5 feet lower and capacity was 1,226 ac.-ft.
5/ Revised from 228,120 to 232,370 ac.-ft.
6/ Guntersville Dam closed Jan. 19, 1939, reducing sediment contributing area to 5,033 sq. mi.
7/ Multiple-use capacity excluding diked areas.
8/ 1953 survey revised.
9/ Used drainage area below Hales Bar Dam (8,935 sq. mi.).
10/ Sediment contributing area reduced by closing Wheeler Dam, Oct. 3, 1936, to 1,135 sq. mi.
11/ Minus (-) indicates scour (treated as negative sediment).
12/ Uncontrolled drainage area 2,776 sq. mi.
13/ Including 19,000 cu. yd. dredged in 1950.
14/ Year survey ranges were established.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

OHIO RIVER BASIN (POINT PLEASANT TO MADISON)
Kanawha, Big Sandy, Licking, Kentucky, Scioto, and Miami River Basins (Continued)

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
19-6	Ohio Cons. Pond No. 73	Blacklick Creek	Columbus, Ohio	26	26	Nov. 1936	—	4.68	—	—	—	—	SCS
	do	do	do	—	—	June 1939	2.6	2.53	—	—	0.035	—	
19-7	Ohio Cons. Pond No. 74	do	do	23	23	Jan. 1937	—	1.04	—	—	—	—	SCS
	do	do	do	—	—	Nov. 1939	2.8	.52	—	—	.008	—	
19-8a	Lake White	Pee Dee Creek	Waverly, Ohio	37.4	36.9	Oct. 1935	—	3,734	—	—	—	—	ODW
	do	do	do	—	—	Dec. 1947	12.0	3,338	—	—	0.894	—	
	do	do	do	—	—	Aug. 1951	4.0	1/3,706	—	68.0	1.73	—	
19-9	Herrington Lake	Dix River	Harrodsburg, Ky.	437	431	Oct. 1925	—	258,969	—	—	—	—	SCS
	do	do	do	—	—	Oct. 1941	16	255,706	—	—	.471	—	
19-10	Walton	Bank Lick Creek	Walton, Ky.	.25	.24	— 1931	—	128	—	—	—	—	SCS
	do	do	do	—	—	— 1937	6	122	—	*50	4.46	4,860	
19-11	Williamstown	Grassy Creek	Williamstown, Ky.	.47	.47	— 1930	—	113	—	—	—	—	SCS
	do	do	do	—	—	— 1937	7	106	—	—	2.13	—	
19-12	Byllesby	New River	Byllesby, Va.	1,310	1,310	Aug. 1912	—	8,092	—	—	—	—	SCS
	do	do	do	—	—	May 1936	23.66	3,538	—	—	2/.184	—	
19-13a	O'Shaughnessy	Scioto River	Dublin, Ohio	980	977	Fall 1925	—	3/16,673	*.026	—	—	—	ODW
	do	do	do	—	—	— 1934 4/	9	15,604	*.024	*65	.121	171	
	do	do	do	—	—	— 1942 4/	8	14,538	*.023	*65	.135	191	
	do	do	do	—	—	July 1951	9	14,162	*.022	44.2	.083	60.5	
19-14a	Becker Lake	Patterson Run	Piqua, Ohio	2.33	2.30	— 1940	—	115	*.084	—	—	—	ODW
	do	do	do	—	—	June 1950	10	94	*.069	51.98	.923	1,032	
19-15b	Madison Lake	Deer Creek	London, Ohio	57.2	57.0	— 1946	—	594.0	—	—	—	—	ODW
	do	do	do	—	—	June 1949	2.6	530.2	—	49.0	.43	459	
	do	do	do	—	—	June 1951	2.0	511.0	—	45.9	.17	170	
	do	do	do	—	—	June 1954	3.0	486.7	—	49.9	.14	152	
19-16b	Grant Lake	Sterling Run	Mt. Orab, Ohio	25.25	24.97	Dec. 1948	—	1,140	.0964	—	—	—	ODW
	do	do	do	—	—	July 1950	1.6	1,111	.0950	86.6	.721	1,360	
	do	do	do	—	—	Sept. 1957	7.0	1,066	.0929	—	.267	—	
	do	do	do	—	—	Aug. 1962	5.0	1,027	.0908	—	.324	—	
	do	do	do	—	—	July 1971	9.0	965	.0870	—	.274	—	
19-17	Caldwell Lake	Trib. of Stoney Creek	Waverly, Ohio	1.02	1.00	— 1937	—	88	*.126	—	—	—	ODW
	do	do	do	—	—	Sept. 1949	12	85	*.120	*60	.253	331	
19-18	Stewart Lake	do	Chillicothe, Ohio	.31	.30	— 1939	—	76	—	—	—	—	ODW
	do	do	do	—	—	Sept. 1951	12.3	71	—	*60	.87	1,137	
19-19	Pine Lake	Tar Hollow Creek	Gillespieville, Ohio	2.42	2.40	Fall 1938	—	134	.073	—	—	—	ODW
	do	do	do	—	—	Aug. 1950	11.7	123	.067	68.95	.398	595	
19-20a	Kiser Lake	Mosquito Creek	St. Paris, Ohio	8.73	8.15	Jan. 1940	—	3,215	.575	—	—	—	ODW
	do	do	do	—	—	Jan. 1950	10.3	3,010	.539	*50	2.40	2,610	
	do	do	do	—	—	July 1954	4.0	2,929	.526	39.7	2.48	2,144	
	do	do	do	—	—	June 1962	8.0	2,867	.513	—	.95	—	
19-21	Eversole Run (Trib. arm of O'Shaughnessy Res.)	Eversole Run	Dublin, Ohio	13.8	13.7	— 1925	—	304	—	—	—	—	ODW
	do	do	do	—	—	— 1951	26	295	—	55.5	.25	302	
19-22	Allen Lake	Trib. of Silver Creek	London, Ohio	.30	.30	— 1938	—	12	*.043	—	—	—	ODW
	do	do	do	—	—	Aug. 1951	13	10	*.036	52.6	.26	298	
19-23	Sylvan Lake (Lower)	Trib. of Beaver Creek	Vienna, Ohio	1.45	1.39	— 1947	—	275	—	—	—	—	ODW
	do	do	do	—	—	Aug. 1951	3.8	261	—	44.8	2.73	2,664	
19-24	Sylvan Lake (Upper)	do	do	.40	.38	— 1948	—	63	—	—	—	—	ODW
	do	do	do	—	—	Aug. 1951	2.8	59	—	*45	3.53	3,460	
19-25	Hosterman Lake	Trib. of Mad River	Springfield, Ohio	1.59	1.58	Nov. 1938	—	83	—	—	—	—	ODW
	do	do	do	—	—	Aug. 1951	12.7	75	—	66.2	.38	548	
19-26	Reynolds Pond	Trib. of Barren Creek	Bidwell, Ohio	.70	.70	— 1940	—	18	*.036	—	—	—	ODW
	do	do	do	—	—	Aug. 1950	10	15	*.029	57.4	.442	552	
19-27	Mt. Gilead Lake (Upper)	Sams Creek	Mt. Gilead, Ohio	8.55	5/5.90	— 1930	—	28	*.006	—	—	—	ODW
	do	do	do	—	—	Oct. 1948	18	26	*.005	*60	.059	51	
19-28	Schott Pond	Trib. of Big Walnut Creek	Westerville, Ohio	.82	.82	— 1921	—	7.3	—	—	—	—	ODW
	do	do	do	—	—	Aug. 1951	30	4.8	—	57.1	.10	124	
19-29a	Lake Alma	Unnamed	Wellston, Ohio	.81	.70	— 1901	—	774	—	—	—	—	ODW
	do	do	do	—	—	Sept. 1951	50	744	—	55.5	.84	1,013	
19-30	Maple Grove Lake	Sams Creek	Mt. Gilead, Ohio	3.05	3.04	— 1932	—	28.1	*.014	—	—	—	ODW
	do	do	do	—	—	Sept. 1949	17	23.9	*.012	83.2	.079	143	
19-31	Pond Lick Lake	Pond Lick Run	Friendship, Ohio	2.56	2.53	— 1938	—	48	*.024	—	—	—	ODW
	do	do	do	—	—	July 1950	12.3	39	*.019	58.8	.277	355	
19-32	Wolfden Lake	Wolfden Run	do	.60	.60	— 1936	—	25.4	*.053	—	—	—	ODW
	do	do	do	—	—	July 1950	14.3	23.1	*.048	59.92	.272	355	
19-33	Bear Lake	Left Fork Bear Creek	do	.48	.47	— 1935	—	33	*.086	—	—	—	ODW
	do	do	do	—	—	July 1950	15.3	32	*.083	67.02	.196	283	
19-34	McBride Lake	Trib. of Pond Run	do	.41	.41	— 1937	—	13	*.027	—	—	—	ODW
	do	do	do	—	—	July 1950	13.3	11	*.023	36.08	.225	167	

19-35	Roosevelt Lake	Turkey Creek	Friendship, Ohio	16.38	15.76	--	1935	--	101	*.088	--	--	--	ODW
	----do----	----do----	----do----	--	--	July	1950	15.3	67	*.005	61.0	.160	186	
19-36a	Adams Lake	Lick Creek	West Union, Ohio	4.45	4.39	Spring	1947	--	318	.087	--	--	--	ODW
	----do----	----do----	----do----	--	--	July	1950	3.5	304	.083	53.24	.095	1,038	
	----do----	----do----	----do----	--	--	Aug.	1960	10.0	277	.076	--	[illegible]	--	
19-37	Vesuvius Lake	Storms Creek	Ironton, Ohio	10.9	10.7	--	1937	--	1,553	.194	--	--	--	ODW
	----do----	----do----	----do----	--	--	June	1949	12	1,463	.183	*54.0	.097	820	
	----do----	----do----	----do----	--	--	July	1952	3	1,463	.183	54.0	.019	22	
19-38	Pike Lake	Richardson Hollow of Morgan Fork	Bainbridge, Ohio	3.44	3.42	--	1937	--	6/91.4	--	--	--	--	ODW
	----do----	----do----	----do----	--	--	Aug.	1950	13	6/86.5	--	60.2	.11	963	
19-39	Upper Hocking No. 2	Hunters Run	Lancaster, Ohio	1.87	1.87	Apr.	1956	--	7/56.98	--	--	--	--	SCS
	----do----	----do----	----do----	--	--	June	1961	5.0	7/51.02	--	68.237	.637	946.711	
19-40	Jackson Lake	Black Fork River	Oak Hill, Ohio	18.8	18.4	--	1940	--	*1,700	--	--	--	--	ODW
	----do----	----do----	----do----	--	--	--	1951	11	*1,590	--	*60.0	.54	705	
19-41	Loramie Lake	Loramie Creek	Ft. Loramie, Ohio	78.7	77.4	--	1840	--	4,115	.099	--	--	--	ODW
	----do----	----do----	----do----	--	--	--	1960	120	3,115	.075	*65	.11	156	
19-42a	Sharon Woods Reservoir	Sharon Creek	Sharonville, Ohio	4.88	4.82	--	1939	--	289	--	--	--	--	ODW
	----do----	----do----	----do----	--	--	--	1942	2.3	274	--	*70	1.27	1,936	
	----do----	----do----	----do----	--	--	--	1945	3.6	232	--	*70	2.44	3,720	
	----do----	----do----	----do----	--	--	--	1949	4.0	206	--	*70	1.38	2,104	
19-43	Cowan Lake	Cowan Creek	Wilmington, Ohio	49.6	48.5	--	1947	--	13,305	.367	--	--	--	ODW
	----do----	----do----	----do----	--	--	--	1960	13	12,770	.352	*90	.847	923	
19-44	Hargus Lake	Hargus Creek	Circleville, Ohio	6.60	6.40	--	1956	--	2,926	.671	--	--	--	ODW
	----do----	----do----	----do----	--	--	--	1960	4	2,892	.663	*90	1.33	1,450	
19-45	Hillsboro	Trib. of Clear Creek	Hillsboro, Ohio	.73	.69	--	1946	--	279.9	.503	--	--	--	ODW
	----do----	----do----	----do----	--	--	--	1955	9	271.4	.487	*40	1.36	1,190	
19-46	Delaware	Olentangy River	Delaware, Ohio	381	379	Mar.	1951	--	132,000	.534	--	--	--	CE
	----do----	----do----	----do----	--	--	May	1960	9.25	8/131,483	.532	*65	8/.1475	8/208	
19-47	W. Fk. Mill Creek Reservoir (Winton Lake)	W. Fork Mill Creek	Cincinnati, Ohio	29.5	28.6	Dec.	1952	--	9/11,466	.609	--	--	--	CE
	----do----	----do----	----do----	--	--	May	1961	8.42	11,207	.595	71.8	1.08	1,489	
19-48	Brush Creek Reservoir	Brush Creek	Butlerville, Ind.	13.4	13.11	Dec.	1957	--	2,067.4	.183	--	--	--	SCS
	----do----	----do----	----do----	--	--	July	[illegible]	9.6	1,883.0	.167	48.0	1.47	1,536.8	
19-49	Whitewater Lake	Silver Creek	Liberty, Ind.	19.29	19.0	Sept.	1954	--	3,590.0	*.249	--	--	--	SCS
	----do----	----do----	----do----	--	--	July	1963	8.8	3,415.6	*.237	67.5	1.04	1,529	
19-50	Jisco Lake	Trib. of Little Salt Creek	Jackson, Ohio	1.67	1.58	--	1952	--	1,109	.860	--	--	--	ODW
	----do----	----do----	----do----	--	--	Sept.	1968	16	1,092	.846	43.7	.67	638	
	----do----	----do----	----do----	--	--	Aug.	1970	2	1,088	.843	50.7	1.16	2,108	
19-51	Clark Lake	Sinking Creek	Vienna, Ohio	6.88	6.72	Late	1957	--	507.3	.106	--	--	--	ODW
	----do----	----do----	----do----	--	--	Sept.	1961	3.8	485.2	.102	--	.865	--	
	----do----	----do----	----do----	--	--	July	1968	7	456.4	.096	--	.612	--	
19-52	Acton Lake	Four Mile Creek	Oxford, Ohio	107.4	106.5	Feb.	1957	--	9,607	.134	--	--	--	ODW
	----do----	----do----	----do----	--	--	Aug.	1961	10/5	9,393	.131	--	.402	--	
19-53	Middle Fork Reservoir	M. Fork of E. Fork White Water Reservoir	Richmond, Ind.	48.14	47.87	Jan.	1962	--	3,250	.101	--	--	--	SCS
	----do----	----do----	----do----	--	--	Oct.	1967	5.5	3,040.71	.095	49.6	.79	850	
19-54	Colonial Mine Lake	Lambs Creek	Beulah, Ky.	1.05	1.00	Fall	1947	--	206.77	.227	--	--	--	SCS
	----do----	----do----	----do----	--	--	May	1967	20	193.93	.214	*70	.642	978.79	
19-55	"A" Strip Mine Lake	Clear Creek D.A.	Richland, Ky.	.009	.009	--	1947	--	53	3.48	--	--	--	SCS
	----do----	----do----	----do----	--	--	Nov.	1967	20	49.3	3.42	*80	20.56	35,824	

TENNESSEE RIVER BASIN (ABOVE HALES BAR DAM)

20-1b	South Holston	South Fork Holston River	Bristol, Tenn.	703	691	Nov.	1950	--	11/661,000	.905	--	--	--	TVA
	----do----	----do----	----do----	--	--	June	1958	7.6	660,581	.903	*55	.249	298	
	----do----	----do----	----do----	--	--	Aug.	1964	6.2	657,963	.900	*55	.611	732	
20-2b	Watauga	Watauga River	Hampton, Tenn.	468	458	Dec.	1948	--	11/572,600	1.159	--	--	--	TVA
	----do----	----do----	----do----	--	--	Mar.	1953	4.3	571,402	1.156	*55	.609	730	
	----do----	----do----	----do----	--	--	Apr.	1958	5.0	570,100	1.154	*55	.568	680	
	----do----	----do----	----do----	--	--	Aug.	1964	6.4	568,839	1.151	*55	.430	515	
20-3b	Boone	South Fork Holston River	Kingsport, Tenn.	1,840	662	Dec.	1952	--	196,668	--	--	--	--	TVA
	----do----	----do----	----do----	--	--	June	1958	5.5	195,633	--	*55	.284	340	
	----do----	----do----	----do----	--	--	Sept.	1964	6.3	193,446	--	*55	.520	623	
20-4b	Fort Patrick Henry	----do----	----do----	1,903	62	Oct.	1953	--	27,110	--	--	--	--	TVA
	----do----	----do----	----do----	--	--	June	1959	5.6	26,945	--	*55	.468	561	
	----do----	----do----	----do----	--	--	Nov.	1964	5.5	26,941	--	*55	.012	14.4	
20-5a	Cherokee	Holston River	Jefferson City, Tenn.	3,428	12/1,477	Dec.	1941	--	1,559,570	--	--	--	--	TVA
	----do----	----do----	----do----	--	--	Apr.	1949	7.4	1,550,932	--	*55	.340	407	
	----do----	----do----	----do----	--	--	Apr.	1954	5.0	1,547,776	--	*55	--	12/--	
	----do----	----do----	----do----	--	--	June	1959	5.1	1,545,960	--	*55	.241	289	
	----do----	----do----	----do----	--	--	June	1964	5.0	1,543,927	--	*55	.276	331	

1/ 1951 survey includes several additional ranges. Results of 1947 and 1951 surveys are not directly comparable.

2/ Based on total sediment in Washington Mill, Buck, Byllesby, and Fields Reservoirs; total drainage area 1,320 sq. mi.

3/ Without 3 ft. flashboards added in 1945.

4/ Survey dates: Sept.-Nov. 1934; Sept.-Dec. 1942.

5/ 1932 correction for Maple Grove.

6/ Reservoir partially dredged in March-April 1949. Data on amount of material are unreliable. Actual amounts of sediment probably 10 to 25 percent greater than indicated.

7/ Sediment pool only.

8/ Flood pool elevation 947. Ranges resurveyed to elevation 915. No appreciable sediment above elevation 915.

9/ Revised on basis of topographic maps developed for definite project report and sedimentation range data.

10/ 1957 water year included.

11/ Multiple-use storage capacity. Reservoir has greater capacity at spillway crest elevation.

12/ Sediment contributing area reduced from 3,381 sq.mi. to 1,477 sq. mi. by closing of Watauga Dam Dec. 1, 1948; South Holston on Nov. 20, 1950; Boone on Dec. 16, 1952; and Fort Patrick Henry on Oct. 27, 1953. Area above Watauga included in 1949 survey as sediment contributing area.

* Estimated or assumed.

SUMMARY OF

RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
			TENNESSEE RIVER BASIN (ABOVE HALES BAR DAM) (Continued)										
20-6d	Nolichucky	Nolichucky River	Greeneville, Tenn.	1,183	1,182	Fall 1913	—	1/3,100	—	—	—	—	TVA
	—do—	—do—	—do—	—	—	Fall 1925	12	2/21,750	.017	—	—	—	
	—do—	—do—	—do—	—	—	Feb. 1938	12.5	14,450	.011	90	0.252	274	
	—do—	—do—	—do—	—	—	Apr. 1947	9.2	12,010	.009	*90	.224	245	
	—do—	—do—	—do—	—	—	May 1953	6.1	9,850	.008	*90	.299	326	
	—do—	—do—	—do—	—	—	Oct. 1958	5.4	7,760	.006	*90	.327	357	
	—do—	—do—	—do—	—	—	Feb. 1961	2.3	7,470	.006	*90	.107	117	
	—do—	—do—	—do—	—	—	Mar. 1967	6.1	2/3,325	.003	*90	.075	82	
	—do—	—do—	—do—	—	—	Sept. 1970	3.5	2,630	.002	*90	.168	183	
20-7e	Douglas	French Broad River	Sevierville, Tenn.	4,541	2,854	Feb. 1943	—	3/1,511,883	.312	—	—	—	TVA
	—do—	—do—	—do—	—	—	July 1949	6.4	1,496,712	.308	*55	.830	994	
	—do—	—do—	—do—	—	—	May 1955	5.8	1,500,870	.309	*55	4/-.251	—	
	—do—	—do—	—do—	—	—	June 1960	5.1	1,489,978	.307	*55	.748	895	
	—do—	—do—	—do—	—	—	Aug. 1967	7.2	1,475,450	.304	*55	.707	847	
20-8e	Fort Loudoun	Tennessee River	Lenoir City, Tenn.	9,550	1,556	Nov. 1946	—	400,965	—	—	—	—	TVA
	—do—	—do—	—do—	—	—	June 1951	4.6	399,414	—	*55	.214	256	
	—do—	—do—	—do—	—	—	Apr. 1956	4.8	398,060	—	*55	.181	217	
	—do—	—do—	—do—	—	—	May 1961	5.1	392,711	—	*55	.674	807	
20-9b	Thorpe	W. Fk. Tuckasegee River	Glenville, N. C.	36.7	34.4	Feb. 1941	—	—	—	—	—	—	TVA
	—do—	—do—	—do—	—	—	Sept. 1950	9.6	3/70,700	.088	—	—	—	
	—do—	—do—	—do—	—	—	July 1958	7.8	70,537	.086	*55	.61	731	
	—do—	—do—	—do—	—	—	May 1969	10.9	70,487	.082	*55	.13	160	
20-10b	Nantahala	Nantahala River	Aquone, N. C.	91	88	Sept. 1950	—	138,083	.561	—	—	—	TVA
	—do—	—do—	—do—	—	—	July 1958	7.9	137,882	.560	*55	.284	340	
	—do—	—do—	—do—	—	—	May 1969	10.8	137,159	.570	*55	.761	911	
20-11e	Fontana	Little Tennessee River	Fontana, N. C.	1,571	1,426	Nov. 1944	—	3/1,455,062	.543	—	—	—	TVA
	—do—	—do—	—do—	—	—	Mar. 1950	5.4	1,452,385	.534	*55	.348	416.9	
	—do—	—do—	—do—	—	—	Sept. 1954	4.5	1,450,646	.534	*55	.271	324.6	
	—do—	—do—	—do—	—	—	June 1959	4.7	1,448,311	.533	*55	.349	418.1	
	—do—	—do—	—do—	—	—	Oct. 1967	8.3	1,443,262	.531	*55	.426	511	
20-12	Cheoah	—do—	Tapoco, N. C.	1,608	1,607	Dec. 1918	—	5/44,030	.016	—	—	—	TVA
	—do—	—do—	—do—	—	—	Sept. 1930	11.8	39,030	.014	*55	.264	316	
	—do—	—do—	—do—	—	—	Aug. 1941	10.9	6/35,030	.013	—	.228	273	
20-13b	Caryville	Cove Creek	Caryville, Tenn.	35.98	35.65	Nov. 1936	—	4/1,726	—	—	—	—	TVA
	—do—	—do—	—do—	—	—	Nov. 1938	2.0	1,652	—	62	1.038	1,401.7	
	—do—	—do—	—do—	—	—	Jan. 1947	8.2	1,580	—	*62	.250	337.6	
	—do—	—do—	—do—	—	—	Mar. 1954	7.1	1,347	—	*62	.917	1,238.3	
	—do—	—do—	—do—	—	—	Aug. 1960	6.6	1,290	—	*62	.250	337.6	
	—do—	—do—	—do—	—	—	July 1970	9.9	1,118	—	*62	.488	659	
20-14e	Norris	Clinch River	Norris, Tenn.	2,912	2,823	Mar. 1936	—	3/7/2,061,569	.692	—	—	—	TVA
	—do—	—do—	—do—	—	—	June 1946	10.3	2,052,357	.691	*55	.317	379.7	
	—do—	—do—	—do—	—	—	June 1954	8.0	2,047,527	.690	*55	.214	256.2	
	—do—	—do—	—do—	—	—	Aug. 1960	6.1	2,044,768	.689	*55	.160	191.7	
	—do—	—do—	—do—	—	—	June 1970	9.8	2,036,324	.686	*55	.305	366	
20-15e	Watts Bar	Tennessee River	Spring City, Tenn.	17,310	2,925	Oct. 1946	—	1,195,229	—	—	—	—	TVA
	—do—	—do—	—do—	—	—	June 1951	4.7	1,193,106	—	*55	.155	186	
	—do—	—do—	—do—	—	—	Apr. 1956	4.8	1,183,735	—	*55	.667	779	
	—do—	—do—	—do—	—	—	June 1961	5.1	1,174,954	—	*55	.589	706	
20-16b	Chatuge	Hiwassee River	Hayesville, N. C.	189	178	Feb. 1942	—	262,325	.705	—	—	—	TVA
	—do—	—do—	—do—	—	—	Aug. 1949	7.5	262,032	.704	*55	.219	262.3	
	—do—	—do—	—do—	—	—	Aug. 1954	5.0	261,502	.702	*55	.595	712.8	
	—do—	—do—	—do—	—	—	Sept. 1960	6.0	260,999	.701	*55	.472	565.4	
	—do—	—do—	—do—	—	—	Apr. 1965	4.6	260,516	.699	*55	.590	707.0	
20-17b	Nottely	Nottely River	Blairsville, Ga.	214	207	Jan. 1942	—	3/176,521	.586	—	—	—	TVA
	—do—	—do—	—do—	—	—	Aug. 1949	7.6	175,865	.584	*55	.415	497.1	
	—do—	—do—	—do—	—	—	Apr. 1955	5.7	174,969	.581	*55	.758	908.0	
	—do—	—do—	—do—	—	—	May 1961	6.0	174,629	.579	*55	.435	521.0	
	—do—	—do—	—do—	—	—	Apr. 1965	4.0	174,357	.579	*55	.087	104.0	
20-18b	Hiwassee	Hiwassee River	Murphy, N. C.	968	8/539	Feb. 1940	—	3/439,741	—	—	—	—	TVA
	—do—	—do—	—do—	—	—	Aug. 1947	7.5	436,009	—	*55	8/—	8/—	
	—do—	—do—	—do—	—	—	May 1953	5.8	435,606	—	*55	.104	125	
	—do—	—do—	—do—	—	—	June 1958	5.0	434,243	—	*55	.536	642	
	—do—	—do—	—do—	—	—	May 1965	6.9	433,568	—	*55	.182	218	
20-19e	Apalachia	—do—	Farner, Tenn.	1,018	48	Feb. 1943	—	58,312	—	—	—	—	TVA
	—do—	—do—	—do—	—	—	Aug. 1950	7.5	57,979	—	*55	.938	1,124	
	—do—	—do—	—do—	—	—	Sept. 1960	10.1	57,550	—	*55	.883	1,058	
	—do—	—do—	—do—	—	—	May 1965	4.7	57,753	—	*55	4/—	—	

20-20c	Blue Ridge	Toccoa River	Blue Ridge, Ga.	232	227	Apr. 1944	—	197,427	.462	—	—	--	TVA
	----do----	----do----	----do----	--	—	Aug. 1949	5.3	196,080	.454	*55	1.119	1,340.5	
	----do----	----do----	----do----	--	--	Aug. 1954	5.0	195,981	.454	*55	.881	1,055.35	
	----do----	----do----	----do----	--	--	June 1959	4.8	196,522	.455	*55	4/-.498	—	
	----do----	----do----	----do----	—	--	Apr. 1968	8.8	195,908	.454	*55	.308	368	
20-21d	Ocoee No. 3	Ocoee River	Ducktown, Tenn.	496	263	Aug. 1942	—	14,304	.017	—	—	—	TVA
	----do----	----do----	----do----	--	--	July 1945	2.9	12,140	--	*64	9/2.114	9/2,947	
	----do----	----do----	----do----	—	—	Nov. 1946	1.3	11,349	—	*64	2.312	3,223	
	----do----	----do----	----do----	—	—	Aug. 1948	1.8	10,570	--	64	1.646	2,294	
	----do----	----do----	----do----	—	—	Aug. 1950	2.0	9,849	—	*64	1.373	1,914	
	----do----	----do----	----do----	—	—	July 1953	3.0	8,696	—	*64	1.460	2,035	
	----do----	----do----	----do----	—	--	Oct. 1955	2.1	8,042	—	*64	1.183	1,649	
	----do----	----do----	----do----	—	--	Oct. 1958	3.0	6,766	—	*64	1.616	2,253	
	----do----	----do----	----do----	--	—	Oct. 1960	2.0	5,920	—	*64	1.608	2,242	
	----do----	----do----	----do----	--	--	June 1962	1.8	5,286	—	*64	1.388	1,865	
	----do----	----do----	----do----	--	--	Sept. 1965	3.2	4,653	—	*64	.753	1,050	
	----do----	----do----	----do----	--	--	Mar. 1968	2.6	4,026	.005	*64	.916	1,278	
20-22d	Ocoee No. 1	----do----	Parksville, Tenn.	595	10/96	Dec. 1911	—	11/109,200	--	—	—	--	TVA
	----do----	----do----	----do----	--	—	Oct. 1940	28.8	12/91,300	—	*64	—	--	
	----do----	----do----	----do----	--	—	Sept. 1949	8.9	13/86,737	—	64	—	—	
	----do----	----do----	----do----	—	—	Aug. 1954	5.0	87,267	—	*64	4/-1.10	—	
	----do----	----do----	----do----	—	--	June 1959	4.9	86,809	—	*64	.974	1,358	
	----do----	----do----	----do----	—	—	Apr. 1968	8.8	86,466	.091	*64	.406	566	
20-23c	Chickamauga	Tennessee River	Chattanooga, Tenn.	20,790	14/1,805	Nov. 1940	--	746,951	—	—	—	—	TVA
	----do----	----do----	----do----	—	—	July 1947	6.7	734,970	--	*60	14/—	14/--	
	----do----	----do----	----do----	—	—	Aug. 1954	7.0	745,178	--	*60	4/-0.808	—	
	----do----	----do----	----do----	--	—	May 1956	1.8	740,367	—	*60	1.481	1,935	
	----do----	----do----	----do----	--	--	June 1961	5.1	738,820	—	*60	.168	220	
20-24d	Hales Bar	----do----	Jasper, Tenn.	21,790	15/990	Oct. 1935	—	153,483	—	—	—	—	TVA
	----do----	----do----	----do----	—	—	Oct. 1940	5.0	152,928	—	61	15/--	--	
	----do----	----do----	----do----	--	--	July 1947	6.7	152,251	—	*61	.102	135	
	----do----	----do----	----do----	--	—	Aug. 1954	7.1	152,992	—	*61	4/-.105	—	
	----do----	----do----	----do----	—	—	May 1956	1.8	153,032	—	*61	4/-.022	—	
	----do----	----do----	----do----	--	—	June 1961	5.1	154,002	—	*61	4/-.192	—	
	----do----	----do----	----do----	—	--	Dec. 1967	6.5	154,012	.0056	—	4/-	—	
20-25	Kanuga Lake	Little Mud Creek	Hendersonville, N. C.	1.54	1.50	— 1908	—	16/246	.120	--	—	—	SCS
	----do----	----do----	----do----	—	—	Oct. 1956	48	190	.093	43.1	.780	732	
20-26	Osceola Lake	Shepard Creek	----do----	4.45	4.39	— 1923	—	16/329	.055	--	--	—	SCS
	----do----	----do----	----do----	--	--	Oct. 1956	33	267	.045	45.7	.428	426	
20-27	Melton Hill	Clinch River	Clinton, Tenn.	3,343	422	July 1963	--	125,900	.0429	—	--	—	TVA
	----do----	----do----	----do----	—	—	May 1970	7	125,578	.0428	*55	.109	131	
20-28	Upper Ollis Creek Reservoir	Ollis Creek	La Follette, Tenn.	10.8	10.75	Apr. 1964	—	561	.051	--	--	--	TVA
	----do----	----do----	----do----	—	—	Oct. 1970	6.5	649	.050	*62	.172	232	

OHIO RIVER BASIN (ABOVE POINT PLEASANT) AND LAKE ERIE DRAINAGE

21-1	Pleasant Hill	Clear Fork	Perrysville, Ohio	199	195	Oct. 1938	—	87,700	.676	—	—	—	CE
	----do----	----do----	----do----	—	—	Feb. 1945	6.25	87,424	.674	*65	.227	321	
21-2a	Senecaville	Seneca Fork	Senecaville, Ohio	121	115	Oct. 1936 17/	--	88,500	.962	—	—	—	CE
	----do----	----do----	----do----	—	—	Mar. 1945	8.3	87,667	.953	*65	.89	1,260	
21-3a	Charles Mill	Black Fork, Mohican River	Mansfield, Ohio	216	207	June 1938	--	88,000	*.657	—	—	—	CE
	----do----	----do----	----do----	--	—	Sept. 1946	8.25	85,886	*.641	*65	1.24	1,755	
	----do----	----do----	----do----	—	—	July 1954	7.75	85,483	*.638	*65	.253	358	
21-4	Hinckston Run	Hinckston Run	Johnstown, Pa.	10.75	10.57	— 1905	—	3,453	—	—	--	—	SCS
	----do----	----do----	----do----	—	—	Sept. 1937	32	3,315	--	*60	.408	533	
21-5	Quemahoning	Quemahoning Creek	----do----	92	90.7	Jan. 1912	—	35,295	--	—	--	—	SCS
	----do----	----do----	----do----	—	—	Sept. 1937	25.8	34,413	—	—	.377	—	
21-6	Salt Lick	Salt Lick Run	----do----	11.86	11.76	-- 1914	--	2,492	—	--	—	--	SCS
	----do----	----do----	----do----	—	--	Sept. 1937	23	2,432	—	*60	.22	287	
21-7a	Bridgeport (Upper)	Jacobs Creek	Mt. Pleasant, Pa.	18/31.64	18/31.52	Mar. 1887	—	642.0	.016	—	—	—	SCS
	----do----	----do----	----do----	--	--	Nov. 1937	50.6	453.0	.011	—	18/.118	168	
	----do----	----do----	----do----	—	—	Nov. 1964	27.0	338.5	.008	65.4	.134	191	

1/ Prior to 1925 dam was 35 ft. lower.

2/ Original volume computed from probings obtained Feb. 1938. Sediment deposits and water inflow computed from dam closure in 1913. Five foot flash boards removed Sept. 1964.

3/ Revised.

4/ Minus (-) indicates scour. Treated as negative sediment.

5/ Computed from probings and available maps.

6/ From first accurate area-volume curves, 1941 conditions.

7/ Multiple-use storage capacity. Reservoir has greater capacity at spillway crest elevation.

8/ Nottely Dam closed Jan. 24, 1942. Chatuge Dam closed Feb. 12, 1942.

9/ Does not include 553 ac.-ft. of sediment flushed into reservoir when small upstream dam failed in Mar. 1944.

10/ Blue Ridge Reservoir closed Dec. 6, 1930. Ocoee #3 closed Aug. 15, 1942.

11/ Based on Tenn. Electric Power Co. data adjusted to latest topographic maps.

12/ Volume computed by average end area method from first accurate area-volume curves - 1940 conditions.

13/ Volume computed from contours based on 1949 survey.

14/ Sediment contributing area reduced by closing Watts Bar Dam on Jan. 1, 1942, and Apalachia Dam, Feb. 14, 1943.

15/ Norris Dam closed Mar. 4, 1936, and Chickamauga Dam closed Jan. 15, 1940, reducing sediment contributing area to 990 sq. mi.

16/ Determined during 1956 survey.

17/ Date storage began. Sediment ranges established and surveyed in Dec. 1937 by Soil Conservation Service, USDA.

18/ Original data revised. 1964 survey data based on corrected elevation and topography from transit survey (1963).

* Estimated or assumed.

SUMMARY OF

RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
	OHIO RIVER BASIN (ABOVE POINT PLEASANT) AND LAKE ERIE DRAINAGE (Continued)												
21-8	Barberton	Wolf Creek	Barberton, Ohio	28.2	28.0	— 1926	—	2,056	*.101	—	—	—	SCS
	do	do	do	—	—	Dec. 1938	12	1,688	*.083	*55	1.10	1,320	
21-9	Buckeye Lake	S. Fork Licking River	Millersport, Ohio	1/49.2	45.1	— 1832	—	2/19,940	—	—	—	—	SCS
	do	do	do	—	—	July 1939	107	15,790	—	*50	.860	937	
21-10	Leesville	McGuire Creek	Leesville, Ohio	48.0	45.7	Aug. 1936	—	37,400	*1.159	—	—	—	SCS
	do	do	do	—	—	Nov. 1939	3.3	37,390	*1.159	*50	.067	73	
21-11	Muskingum College	Trib. of Muskingum River	New Concord, Ohio	.32	.31	— 1915	—	9.41	*.062	—	—	—	SCS
	do	do	do	—	—	Nov. 1935	20	7.05	*.031	*70	.381	581	
21-12	Ohio Cons. Pond No. 22	Trib. of Duck Creek	Marietta, Ohio	.05	.05	Oct. 1938	—	4.43	*.116	—	—	—	SCS
	do	do	do	—	—	Nov. 1939	1.1	4.21	*.110	*40	4.00	3,485	
21-13	Ohio Cons. Pond No. 51	Trib. of Ohio River	do	.13	.13	Oct. 1938	—	15.16	*.152	—	—	—	SCS
	do	do	do	—	—	Nov. 1939	1.1	15.02	*.151	*40	.977	851	
21-14	Ohio Cons. Pond No. 52	do	do	.17	.16	— 1928	—	17.95	*.138	—	—	—	SCS
	do	do	do	—	—	Nov. 1939	11	15.40	*.118	*50	1.45	1,579	
21-15	Robbins Lake	Ford Creek	Hartford, Ohio	4.47	4.45	Summer 1922	—	91.6	*.028	—	—	—	SCS
	do	do	do	—	—	Dec. 1939	17.4	79.9	*.025	*68	.151	224	
21-16	Gorleys Lake	Trib. of Youghiogheny R.	Uniontown, Pa.	3.00	2.97	— 1908	—	150	—	—	—	—	SCS
	do	do	do	—	—	June 1939	30	138	—	*60	.135	176	
21-17	Lake Rockwell	Cuyahoga River	Kent, Ohio	205.5	3/126.1	Aug. 1914	—	7,423	—	—	—	—	SCS
	do	do	do	—	—	Aug. 1950	36	6,887	—	46	.120	120	
21-18a	Stony Lake	McGuire Creek	Perrysville, Ohio	11.75	11.72	— 1927	—	121	—	—	—	—	ODW
	do	do	do	—	—	Dec. 1938	11.0	80	—	*65	.319	452	
	do	do	do	—	—	May 1949	10.4	61	—	*65	.156	221	
21-19	Tabor Club Lake	Small Br. Muskingum River	do	.55	.54	— 1923	—	53.6	*.145	—	—	—	SCS
	do	do	do	—	—	Dec. 1938	15	42.6	*.115	*50	4/1.38	1,500	
21-20	Zanesville Nursery Lake	N. Br. Blount Run	Zanesville, Ohio	2.95	2.93	Dec. 1936	—	76.62	*.034	—	—	—	SCS
	do	do	do	—	—	July 1941	4.6	72.23	*.032	*65	.326	462	
21-21a	Tionesta	Tionesta Creek	Tionesta, Pa.	478	474	Feb. 1941	—	133,400	.210	—	—	—	CE
	do	do	do	—	—	Sept. 1949	8.6	132,571	.209	*65	.203	287	
21-22b	Loyalhanna	Loyalhanna Creek	Saltsburg, Pa.	290	285	June 1942	—	5/95,300	.278	—	—	—	CE
	do	do	do	—	—	Oct. 1948	6.33	94,652	.276	*44	.358	343	
	do	do	do	—	—	Sept. 1953	4.92	94,192	.275	44	.326	312	
	do	do	do	—	—	Apr. 1962	8.58	93,501	.273	*44	.284	272	
21-23b	Mahoning Creek	Mahoning Creek	Dayton, Pa.	340	336	June 1941	—	6/76,200	.180	—	—	—	CE
	do	do	do	—	—	Aug. 1948	7.2	73,807	.172	*65	.162	229	
	do	do	do	—	—	June 1965	16.8	72,800	.177	43	.178	167	
21-24b	Crooked Creek	Crooked Creek	Ford City, Pa.	277	274	Apr. 1940	—	7/93,900	.307	—	—	—	CE
	do	do	do	—	—	Sept. 1945	5.4	93,808	.307	*65	.062	87.8	
	do	do	do	—	—	Aug. 1964	18.9	92,376	.302	51	.277	308	
21-25a	Tygart River	Tygart River	Grafton, W. Va.	1,184	1,179	Oct. 1937	—	289,600	.175	—	—	—	CE
	do	do	do	—	—	Feb. 1945 8/	—	—	—	—	—	—	
	do	do	do	—	—	Mar. 1959	21.5	287,700	.174	*65	.075	106	
21-26a	Youghiogheny River	Youghiogheny River	Confluence, Pa.	434	428	Oct. 1943	—	254,000	.406	—	—	—	CE
	do	do	do	—	—	Oct. 1949	6.0	253,121	.405	*65	.341	483	
21-27a	Atwood Reservoir	Indian Fk., Conotton Creek	Sherrodsville, Ohio	70	66.2	Apr. 1940	—	49,700	.994	—	—	—	CE
	do	do	do	—	—	Dec. 1946	6.7	49,570	.991	*65	.29	410	
21-28	Babb Pond	Unnamed	Richfield, Ohio	.02	.02	— 1932	—	.245	*.019	—	—	—	SCS
	do	do	do	—	—	Apr. 1951	19	.189	*.015	*60	.15	196	
21-29	Bascom Pond	do	Hudson, Ohio	.32	.32	— 1944	—	3.87	*.019	—	—	—	SCS
	do	do	do	—	—	Apr. 1951	7	3.24	*.016	*60	.281	367	
21-30	Christener Pond	do	Parma, Ohio	.09	.09	— 1940	—	3.40	*.098	—	—	—	SCS
	do	do	do	—	—	Apr. 1951	11	2.79	*.068	*60	.611	798	
21-31	Schoenbeck Pond	do	Richfield, Ohio	.03	.03	— 1940	—	1.48	*.076	—	—	—	SCS
	do	do	do	—	—	Apr. 1951	11	1.29	*.066	*60	.567	741	
21-32	Ticky Pond	do	Peninsula, Ohio	.16	.16	— 1949	—	2.84	*.027	—	—	—	SCS
	do	do	do	—	—	Apr. 1951	2	2.81	*.027	*60	.100	131	
21-33b	Lake Hope	Sandy Run	Zaleski, Ohio	9.95	9.75	May 1939	—	1,555	.201	—	—	—	ODW
	do	do	do	—	—	July 1949	10.2	1,511	.195	74.8	.442	720	
	do	do	do	—	—	June 1955	5.9	1,481	.191	—	.523	—	
	do	do	do	—	—	Aug. 1969	14.2	1,451	.187	—	.213	—	
21-34b	Jefferson Lake	Town Fk. Yellow River	Richmond, Ohio	7.52	7.48	Dec. 1935	—	178	.029	—	—	—	ODW
	do	do	do	—	—	Aug. 1950	15	156	.025	77.9	.199	164	
	do	do	do	—	—	June 1956	6	134	.022	—	.466	—	
	do	do	do	—	—	June 1961	5	131	.021	—	.078	—	

21-35a	Clouse Lake	Ctr. Br. Rush Creek	Somerset, Ohio	8.7	8.66	—	1948	—	234	.037	—	—	—	ODNR
	do	do	do	—	—	—	1949	2.0	217	.035	9/55.7	1.00	10/1,213	
	do	do	do	—	—	—	1950	1.0	206	.033	9/50.6	1.27	1,177	
	do	do	do	—	—	—	1951	1.0	202	.032	*9/53	.45	693	
	do	do	do	—	—	—	1952	1.0	199	.032	9/55.6	.34	609	
	do	do	do	—	—	—	1953	1.0	197	.031	9/65.3	.25	1,239	
	do	do	do	—	—	—	1954	1.0	193	.031	9/68.2	.41	926	
	do	do	do	—	—	—	1955	1.0	192	.031	—	.14	—	
	do	do	do	—	—	—	1956	1.0	187	.030	—	.55	—	
	do	do	do	—	—	—	1957	1.0	183	.029	—	.47	—	
	do	do	do	—	—	—	1958	1.0	177	.028	—	.44	—	
	do	do	do	—	—	—	1959	1.0	176	.028	—	.18	—	
	do	do	do	—	—	—	1961	2.0	170	.027	—	.37	—	
	do	do	do	—	—	—	1962	1.0	166	.026	—	.42	—	
	do	do	do	—	—	—	1963	1.0	163	.026	—	.33	—	
	do	do	do	—	—	—	1964	1.0	159	.025	—	.45	—	
	do	do	do	—	—	—	1966	2.0	150	.024	—	.55	—	
	do	do	do	—	—	—	1967	1.0	148	.024	—	.22	—	
	do	do	do	—	—	—	1968	1.0	146	.023	—	.236	—	
	do	do	do	—	—	—	1969	1.0	145	.023	—	.136	—	
	do	do	do	—	—	—	1970	1.0	142	.023	—	.319	—	
21-36	East Branch	E. Br. Cuyahoga River	Burton, Ohio	17.53	16.88	—	1939	—	4,659	*.340	—	—	—	ODNR
	do	do	do	—	—	June	1949	9.7	4,535	*.331	*40	.759	661	
21-37	Westville Lake	Trib. of Mahoning River	Alliance, Ohio	8.36	8.22	—	1913	—	994	—	—	—	—	ODNR
	do	do	do	—	—	Aug.	1950	37	933	—	66	.20	287	
21-38	Berlin	Mahoning River	Berlin Center, Ohio	249	246	July	1943	—	—	—	—	—	—	CE
	do	do	do	—	—	Nov.	1951	8.4	91,200	.551	*100	1.30	2,830	
21-39a	Mountain Lake	Broad Ford Run	Mt. Lake Park, Md.	7.4	7.4	—	1880*	—	122.75	—	—	—	—	SCS
	do	do	do	—	—	July	1957	76	77.02	—	—	.081	—	
21-40	Blaser Lake	Trib. of Sandy Creek	Minerva, Ohio	.07	.07	—	1931	—	.15	—	—	—	—	ODNR
	do	do	do	—	—	—	1938	7	.0	—	*65	.29	410	
21-41	Braden Lake	do	do	.12	.12	—	1930	—	.11	—	—	—	—	ODNR
	do	do	do	—	—	—	1938	8	.00	—	65	.08	113.3	
21-42	Lake Milton	Mahoning River	Milton, Ohio	280	277.4	—	1916	—	28,100	—	—	—	—	ODNR
	do	do	do	—	—	—	1941	25	22,250	—	*65	.84	1,189	
21-43	Peters Pond	Trib. of Indian Creek	Lancaster, Ohio	.11	.11	—	1946	—	9.46	—	—	—	—	ODNR
	do	do	do	—	—	—	1954	8	9.11	—	48.1	.36	377	
21-44	Wyandot Lake	Trib. of Clear Creek	Logan, Ohio	.58	.58	—	1930	—	11.5	—	—	—	—	ODNR
	do	do	do	—	—	—	1948	18	4.65	—	—	.657	—	
21-45	Orchard Park	Pipe Creek	Orchard Park, N. Y.	1.67	—	—	1928	—	169.99	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1951	23	161.26	—	*60	.23	300	
21-46	Terra Alta Lake	Snowy River	Kingwood, W. Va.	4.76	—	—	1902	—	144.8	—	—	—	—	SCS
	do	do	do	—	—	June	1952	50	117.3	—	*50	.117	127	
21-47	Centerville Mills Lake	Aurora Br. Chagrin River	Aurora, Ohio	10.4	10.38	—	1855	—	86.3	.010	—	—	—	ODNR
	do	do	do	—	—	—	1949	94	38.3	.004	*65	.0493	69.6	
21-48	Meander Creek	Meander Creek	Youngstown, Ohio	84.9	81.76	—	1929	—	32,400	.597	—	—	—	ODNR
	do	do	do	—	—	—	1949	20	31,835	.586	*65	.346	489	
21-49	Navard Mason	Salem Fork	Salem, W. Va.	.29	.29	Oct.	1954	—	69.58	.265	—	—	—	SCS
	do	do	do	—	—	Oct.	1956	2.0	68.95	.262	*60	1.09	1,424.4	
	do	do	do	—	—	Oct.	1958	2.0	68.77	.261	—	.31	405.1	
	do	do	do	—	—	Dec.	1960	2.2	68.60	.261	—	.27	352.8	
21-50	Knox Lake	E. Branch Kokosing	Fredericktown, Ohio	31.4	30.66	11/Fall	1949	—	3,502	.146	—	—	—	ODNR
	do	do	do	—	—	Aug.	1960	7	3,399	.142	—	.48	—	
	do	do	do	—	—	July	1970	10	3,283	.137	—	.378	—	
21-51	Veto Lake	Little Hocking River	Belpre, Ohio	20.04	19.79	May	1954	—	1,111	.0642	—	—	—	ODNR
	do	do	do	—	—	July	1955	1.2	1,092	.0631	—	.787	—	
	do	do	do	—	—	July	1960	5	1,059	.0612	—	.341	—	
	do	do	do	—	—	Aug.	1966	6	1,009	.0583	—	.419	—	
21-52	Clear Fork Reservoir	Clear Fork	Lexington, Ohio	33.63	32.05	Oct.	1948	—	13,176	.565	—	—	—	ODNR
	do	do	do	—	—	Aug.	1956	8	12,886	.553	—	1.13	—	
	do	do	do	—	—	July	1962	6	12,731	.546	—	.806	—	
21-53	Findley Lake	Wellington Creek	Wellington, Ohio	6.25	6.12	Mar.	1956	—	760.4	.190	—	—	—	ODNR
	do	do	do	—	—	June	1961	5.6	745.5	.186	—	.435	—	
21-54	Lake Logan	Clear Fork	Logan, Ohio	14.7	14.17	Feb.	1955	—	3,142	.291	—	—	—	ODNR
	do	do	do	—	—	Aug.	1966	11.8	3,081	.285	—	.363	—	
21-55	Ischua Creek Site #2	Johnson Creek	Franklinville, N. Y.	2.8	2.75	Aug.	1964	—	465	.156	—	—	—	SCS
	do	do	do	—	—	July	1967	2.91	464.72	.156	*80	.0345	60.5	
	do	do	do	—	—	Sept.	1969	2.16	464.33	.156	80	.0665	116	
21-56	Ischua Creek Site #5	Gates Creek	do	6.4	6.28	Aug.	1964	—	1,085	.159	—	—	—	SCS
	do	do	do	—	—	Aug.	1967	3	1,083.77	.159	*80	.065	113.3	
	do	do	do	—	—	Sept.	1969	2.12	1,082.95	.159	80	.062	108	

1/ Drainage area has been 115 sq. mi. (net) part of the time in the past, when fed partly by feeder from S. Fork Kirkersville River.

2/ At present spillway elevation (lowered in 1908). From 1836 to 1908, spillway elevation was 893.4, surface area 3,636 acres, and capacity 22,090 ac.-ft. From 1832 to 1836, spillway elevation was lower and surface area was 3,136 acres. Natural lake of 650 acres originally.

3/ East Branch Reservoir and many natural lakes act as efficient sediment traps.

4/ Dam failed earlier in 1938 but little sediment lost.

5/ At elevation 975. Elevation top of gates = 976 ft.

6/ At elevation 1,162. Elevation top of gates = 1,164 ft.

7/ At elevation 920. Spillway crest elevation = 918 ft.

8/ In view of the limited amount of sediment computed in the 1965 report (384 ac.-ft.), the values of the 1945 report are not included and were not used in computing survey data for the 1959 report. It is considered that the longer period (21.5 yr.) was necessary to develop a volume capable of being measured with a reasonable degree of accuracy.

9/ Density for entire period of record.

10/ Computed from differences in total sediment accumulation in tons at each survey.

11/ Gate closed Fall 1949. Drained March 1950. Gate closed May 1954.

* Estimated or assumed.

SUMMARY OF

RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
			OHIO RIVER BASIN (ABOVE POINT PLEASANT) AND LAKE ERIE DRAINAGE (Continued)										
21-57	North Fk. Cowanesque River (PA-406)	White Branch	North Fork, Pa.	3.4	3.3	Aug. 1962	--	654.13	.21	--	--	--	SCS
	do	do	do	--	--	July 1967	4.9	653.89	.21	71	1/0.015	1/23.2	
21-58	Salem Fork No. 11A	Varner Hollow	Salem, W. Va.	.288	.287	Oct. 1954	--	53	.168	--	--	--	SCS
	do	do	do	--	--	Oct. 1956	2	52.25	.166	*60	1.307	1,708	
	do	do	do	--	--	Oct. 1958	2	52.05	.165	--	.348	455	
	do	do	do	--	--	Dec. 1960	2.17	51.95	.165	--	.16	209	
	do	do	do	--	--	June 1962	1.58	51.75	.164	--	.443	579	
21-59	Upper Hocking No. 1	Hunters Run	Lancaster, Ohio	1.04	.94	Apr. 1956	--	490	.882	--	--	--	SCS
	do	do	do	--	--	June 1962	6.08	446.13	.875	71.6	.678	1,057	
			GREAT LAKES DRAINAGE (IN MICHIGAN) AND MAUMEE RIVER BASIN										
22-1	Grand2/	St.Marys and Wabash River	Celina, Ohio	119	93	-- 1844	--	3/130,175	*2.193	--	--	--	SCS
	do	do	do	--	--	Aug. 1940	96	106,605	*1.796	*55	2.64	3,162	
22-2	Stronach	Pine R. of Manistee River	Wellston, Mich.	291	233	-- 1912	--	640	*.003	--	--	--	SCS
	do	do	do	--	--	Jan. 1953	41	27	*.0001	110.4	.064	154	
22-3	Goller Pond	Unnamed	Defiance, Ohio	.026	.024	Mar. 1945	--	9.5	*.761	--	--	--	ODW
	do	do	do	--	--	Aug. 1951	6.4	9.4	*.753	*45	.92	902	
22-4a	Auglaize R. Power	Auglaize River	do	2,329	2,326	-- 1912	--	5/11,400	.012	--	--	--	ODW
	do	do	do	--	--	-- 1951	39	11,600	.009	--	.031	--	
22-5	Eagle Creek5/	Eagle Creek	do	5.23	5.20	-- 1912	--	129	.050	--	--	--	ODW
	do	do	do	--	--	July 1951	39	74	.029	59	.27	347	
22-6	Beetree Creek5/	Beetree Creek	do	1.94	1.91	-- 1912	--	148	*.156	--	--	--	ODW
	do	do	do	--	--	Aug. 1951	39	104	*.109	52.5	.59	675	
22-7	Batt Pond	Unnamed	do	.013	.012	Apr. 1947	--	2.6	*.417	--	--	--	ODW
	do	do	do	--	--	July 1951	4.3	2.5	*.401	73.4	2.75	4,396	
22-8	Harrison Lake	Mill Creek	Fayette, Ohio	37.2	37.0	-- 1941	--	991	.051	--	--	--	ODW
	do	do	do	--	--	June 1949	8.3	929.1	.048	53.2	.20	232	
	do	do	do	--	--	July 1951	2.1	902.4	.046	*53	.34	392	
22-9	Allmandinger Pond	Unnamed	Ohio City, Ohio	.038	.035	Jan. 1945	--	5.08	*.239	--	--	--	ODW
	do	do	do	--	--	July 1951	6.7	4.71	*.221	53.7	1.71	2,001	
22-10	Bucyrus No. 2	Trib. of Sandusky River	Bucyrus, Ohio	2.84	2.79	-- 1919	--	242	.120	--	--	--	ODW
	do	do	do	--	--	June 1949	30	218	.108	45.4	.28	277	
22-11e	Contris Pond6/	Trib. of Hoo Creek	Lafayette, Ohio	.13	.13	-- 1947	--	9.2	.133	--	--	--	ODW
	do	do	do	--	--	-- 1951	4	7.9	.114	57.8	2.60	3,270	
	do	do	do	--	--	-- 1954	3	7.6	.107	*49.3	1.36	1,460	
	do	do	do	--	--	-- 1962	8	6.3	.091	--	.969	--	
22-12	Sixmile Creek5/	Sixmile Creek	Defiance, Ohio	21.6	21.4	-- 1912	--	995	*.094	--	--	--	ODW
	do	do	do	--	--	July 1951	39	696	*.066	43.8	.36	343	
22-13a	Burt Lake	Unnamed	Oakwood, Ohio	.76	.74	Sept. 1948	--	59	*.155	--	--	--	ODW
	do	do	do	--	--	July 1951	2.8	57	*.150	37.2	1.13	924	
22-14	Kohart Pond	Unnamed	Grover Hill, Ohio	.021	.019	Sept. 1943	--	2.4	*.229	--	--	--	ODW
	do	do	do	--	--	July 1951	7.8	2.3	*.219	26.1	.53	301	
22-15	Van Buren Lake	Rocky Ford Creek	Findlay, Ohio	22.80	22.72	Spring 1939	--	248	.022	--	--	--	ODW
	do	do	do	--	--	Nov. 1948	9.5	205	.019	53.6	.20	233	
	do	do	do	--	--	Aug. 1951	2.8	186	.017	57.9	.31	391	
22-16	Norvell Lake	River Raisin	Norvell, Mich.	58.5	25.3	-- --	--	717.6	.026	--	--	--	SCS
	do	do	do	--	--	May 1969	100±	502.3	.018	23.3	.085	43	
22-17	Sharon Hollow	do	Manchester, Mich.	34.7	25	-- 1927	--	298.1	.016	--	--	--	SCS
	do	do	do	--	--	May 1969	42	143.9	.009	30.1	.11	72	
22-18	Brooklyn Mill Pond	do	Brooklyn, Mich.	29.6	6.2	-- 1948	--	249.3	.016	--	--	--	SCS
	do	do	do	--	--	May 1969	21	186.3	.012	43.4	.484	457	
22-19	Manchester Power Dam	do	Manchester, Mich.	6.4	6.4	-- 1945	--	288.9	.09	--	--	--	SCS
	do	do	do	--	--	May 1969	23	259.5	.08	31.5	.20	137	
22-20	Iron Mill Pond	Iron Creek	do	14.4	5.2	-- --	--	1,551	.224	--	--	--	SCS
	do	do	do	--	--	Aug. 1969	100	1,159	.168	--	.75	--	
22-21	Phoenix Pond	Middle River Rouge	Plymouth, Mich.	56.85	56.85	-- --	--	225	.0074	--	--	--	SCS
	do	do	do	--	--	Sept. 1969	100	170	.0056	41.8	.0097	8.8	
22-22	Saline Mill Pond	Saline River	Saline, Mich.	73.6	63	-- 1937	--	240.1	.0061	--	--	--	SCS
	do	do	do	--	--	Mar. 1969	31	129.6	.0033	44	.057	54	
22-23	Manchester Mill Pond	Raisin River	Manchester, Mich.	17	17	-- 1906	--	21.33	.0026	--	--	--	SCS
	do	do	do	--	--	May 1969	63	10.9	.0013	38	.01	8	
22-24	Bridgeway Lake	Unnamed	Dexter, Mich.	7.5	7.5	-- 1927	--	76.7	.0250	--	--	--	SCS
	do	do	do	--	--	Mar. 1969	41	47.9	.0156	46	.093	93	
22-25	Franklin Mill Pond	Franklin Branch Rouge	Franklin, Mich.	13.8	7.8	-- 1833	--	97.8	.0133	--	--	--	SCS
	do	do	do	--	--	Apr. 1969	136	13.1	.0018	50	.079	86	
22-26	Waterford Pond	Middle River Rouge	Northville, Mich.	54	54	-- --	--	173	.0067	--	--	--	SCS
	do	do	do	--	--	Sept. 1969	100	101	.0039	39	.013	11	

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22-27	Tecumseh Mill Pond	Evans Creek	Tecumseh, Mich.	31.6	26.3	—	1827	—	227.8	.01901	—	—	—	SCS
	do	do	do	—	—	Apr.	1969	142	94.7	.0062	59	.0356	45	
22-28	Belleville Lake	Huron River	Belleville, Mich.	810	20.3	—	1929	—	19,945	.0513	—	—	—	SCS
	do	do	do	—	—	July	1969	40	17,980	.0462	69	2.42	3,637	
22-29	Ford Lake	do	Ypsilanti, Mich.	790	11.2	—	1933	—	17,926	.0472	—	—	—	SCS
	do	do	do	—	—	July	1969	36	16,085	.0424	81	4.56	8,045	
22-30	Barton Pond	do	Ann Arbor, Mich.	708	183	—	1915	—	3,150	.0083	—	—	—	SCS
	do	do	do	—	—	July	1969	54	2,601	.0068	39	.055	46	
22-31	Red Mill Pond	Upper Raisin	Tecumseh, Mich.	172.9	25.9	—	—	—	677	.0081	—	—	—	SCS
	do	do	do	—	—	Aug.	1969	100+	336	.0040	41.9	.13	118	
22-32	H. H. Fry Pond	Squaw Creek	Onsted, Mich.	12.45	12.45	Early	1962	—	121.3	.020	—	—	—	SCS
	do	do	do	—	—	Aug.	1969	7	116	.019	—	.06	—	
22-33	Newburgh Lake	Middle Rouge	Plymouth, Mich.	54.3	54.3	—	1933	—	667.8	.026	—	—	—	SCS
	do	do	do	—	—	Sept.	1969	36	562.9	.022	56.08	.054	65.96	
22-34	Lake Adrian	Wolf Creek	Adrian, Mich.	65	59	—	1942	—	1,000	.032	—	—	—	SCS
	do	do	do	—	—	Sept.	1969	28	851	.027	75.1	.09	147	

GREAT LAKES DRAINAGE (IN MICHIGAN AND WISCONSIN)

23-

MISSISSIPPI RIVER BASIN (LOUISIANA TO CHESTER) ILLINOIS, KASKASKIA, AND MERAMEC RIVER BASINS

24-1a	Lake Williamson (Artic Pond)	Trib. of Honey Creek	Carlinville, Ill.	.53	.51	—	1922	—	175.6	.614	—	—	—	IWS
	do	do	do	—	—	—	1949	27	159.5	.558	46.24	1.18	1,188	
	do	do	do	—	—	—	1954	5	152.2	.532	46.24	2.90	2,921	
	do	do	do	—	—	—	1961	7	147.6	.516	46.24	1.25	1,259	
24-2b	Lake Bloomington	Money Creek	Hudson, Ill.	61	60	Dec.	1929	—	6,654	1.981	—	—	—	IWS
	do	do	do	—	—	Aug.	1948	18.7	6,062	1.805	41.5	.528	477	
	do	do	do	—	—	Aug.	1952	4.0	5,905	1.756	41.5	.655	592	
	do	do	do	—	—	July	1955	2.9	5,863	1.746	46	.242	243	
24-3b	Lake Carlinville	Honey Creek	Carlinville, Ill.	26.06	25.79	June	1939	—	1,725	.118	—	—	—	IWS
	do	do	do	—	—	June	1949	10.4	1,475	.101	59.1	.934	1,200	
	do	do	do	—	—	July	1954	5.1	1,427	.098	65.7	.365	521	
	do	do	do	—	—	Sept.	1959	5.1	1,387	.095	62.4	.302	480	
24-4a	Lake Decatur	Sangamon River	Decatur, Ill.	906	902	Apr.	1922	—	19,738	—	—	—	—	SCS
	do	do	do	—	—	July	1936	14.2	16,930	—	—	.220	248	
	do	do	do	—	—	June	1946	10.0	14,567	—	51.7	.262	295	
24-5	Shaefer Pond	Trib. of Cahokia Creek	Edwardsville, Ill.	.087	.083	Nov.	1937	—	20.2	—	—	—	—	IWS
	do	do	do	—	—	July	1949	11.8	17.6	—	*50	2.65	2,890	
24-6	Lake Springfield	Sugar and Lick Creeks	Springfield, Ill.	265	258	Jan.	1934	—	61,039	—	—	—	—	IWS
	do	do	do	—	—	Aug.	1948	14.6	58,380	—	43	.705	660	
24-7	Spring Lake	Spring Creek	Macomb, Ill.	20.2	20.1	Apr.	1927	—	607	—	—	—	—	IWS
	do	do	do	—	—	Sept.	1947	20.4	320	—	59.7	.701	911	
24-8	Lake Bracken	Brush Creek	Galesburg, Ill.	9.14	8.85	Dec.	1923	—	2,881	—	—	—	—	IWS
	do	do	do	—	—	Aug.	1936	12.7	2,660	—	—	1.97	2,231	
	do	do	do	—	—	June	1949	12.9	2,452	—	52	1.82	2,061	
24-9	Pittsfield	Trib. of Panther Creek	Pittsfield, Ill.	1.84	1.77	June	1925	—	7/367	*.490	—	—	—	SCS
	do	do	do	—	—	Dec.	1946	21.5	232	*.310	*40	3.55	3,090	
24-10a	Lock and Dam 25 (Winfield)	Mississippi River	Winfield, Mo.	142,000	—	July	1939	—	180,000	—	—	—	—	CE
	do	do	do	—	—	May	1945	5.9	194,100	—	—	—	—	
	do	do	do	—	—	May	1946	1.0	181,400	—	—	—	—	
	do	do	do	—	—	Dec.	1947	1.6	184,000	—	—	—	—	
	do	do	do	—	—	May	1959	11.4	155,000	—	—	—	—	
24-11a	Mt. Sterling	Trib. of Shelby Creek	Mt. Sterling, Ill.	1.80	1.75	—	1935	—	306	.393	—	—	—	IWS
	do	do	do	—	—	—	1951	16	248	.319	60	2.05	2,679	
24-12	Lake Jacksonville	Sandy Creek	Jacksonville, Ill.	10.8	10.1	—	1939	—	7,058	1.167	—	—	—	IWS
	do	do	do	—	—	June	1952	12.0	6,874	1.137	32.5	1.51	1,069	
24-13	Mauvaisse Terre Lake	Mauvaisse Terre Creek	do	32.6	32.2	—	1921	—	1,820	.100	—	—	—	IWS
	do	do	do	—	—	June	1952	31	1,215	.067	48.2	.61	640	
24-14a	Morgan Lake	Unnamed	do	2.75	2.72	—	1900	—	126	.082	—	—	—	IWS
	do	do	do	—	—	June	1952	52	73	.047	41.2	.38	341	
24-15a	Langdon Pond	do	Franklin, Ill.	.358	.348	—	1907	—	56.8	.284	—	—	—	IWS
	do	do	do	—	—	July	1952	45	44.8	.224	31.8	.78	540	
24-16	Franklin Outing Club Lake	do	do	.450	.414	—	1905	—	328	1.302	—	—	—	IWS
	do	do	do	—	—	July	1952	47	301	1.194	52.4	1.43	1,632	
24-17	Anderson Lake	do	Concord, Ill.	.631	.600	—	1909	—	267	.756	—	—	—	IWS
	do	do	do	—	—	July	1952	43	234	.663	36.2	1.28	1,009	
24-18	Waverly City	do	Waverly, Ill.	9.24	9.16	Oct.	1938	—	308.3	.060	—	—	—	IWS
	do	do	do	—	—	July	1952	13.8	238.6	.046	42.4	.551	509	
24-19	Whitehall City	do	Whitehall, Ill.	.97	.92	—	1897	—	459	.846	—	—	—	IWS
	do	do	do	—	—	July	1952	55	408	.751	43.1	1.02	957	
24-20	Roodhouse Park District Lake	do	Roodhouse, Ill.	.451	.439	—	1917	—	61.6	.243	—	—	—	IWS
	do	do	do	—	—	July	1952	35	53.9	.213	42.4	.50	462	
24-21	Woodbine Country Club Lake	do	Greenfield, Ill.	.329	.320	—	1926	—	58.5	.318	—	—	—	IWS
	do	do	do	—	—	July	1952	26	43.4	.236	54.4	1.81	2,144	
24-22	Dale Cole Pond	do	do	.231	.221	—	1924	—	67.2	.521	—	—	—	IWS
	do	do	do	—	—	July	1952	28	56.7	.440	53.7	1.72	2,012	
24-23	Sealy Pond	do	Hillview, Ill.	.093	.091	—	1902	—	3.7	.071	—	—	—	IWS
	do	do	do	—	—	July	1952	50	2.2	.042	35.6	.30	233	

1/ Sediment pool only.
2/ Also known as Lake St. Marys.
3/ At present spillway elevation (lowered in 1856). In 1844 surface area was 17,603 acres and storage capacity was 220,400 ac.-ft.
4/ Capacity and sediment figures include that of several tributary arms including Beetree, Eagle and Sixmile Creeks.
5/ Tributary arm of Auglaize River Power Reservoir.
6/ All figures subject to readjustment because of minor dredging which occured in 1953.
7/ At present spillway elevation; spillway raised 1.24 ft. in 1944.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
	MISSISSIPPI RIVER BASIN (LOUISIANA TO CHESTER) ILLINOIS, KASKASKIA, AND MERAMEC RIVER BASINS (Continued)												
24-24	Vineyard Pond	Unnamed	Whitehall, Ill.	0.054	0.052	Jan. 1937	--	1.58	.053	--	--	--	IWS
	do	do	do	--	--	July 1952	15.6	1.32	.044	36.2	0.33	260	
24-25	Kampp	do	Springfield, Ill.	3.49	3.43	-- 1907	--	181.8	.093	--	--	--	IWS
	do	do	do	--	--	Aug. 1952	45	115.3	.059	38.7	.431	363	
24-26	Aschauer Pond	do	Riverton, Ill.	.526	.518	-- 1939	--	18.3	.062	--	--	--	IWS
	do	do	do	--	--	Aug. 1952	13	9.4	.032	56.2	1.31	1,603	
24-27	Schmidt Pond	do	Chatham, Ill.	1.31	1.30	-- 1943	--	6.0	.008	--	--	--	IWS
	do	do	do	--	--	Aug. 1952	9	3.4	.005	41.2	.22	197	
24-28	Ayers	Trib. of Shoal Creek	Ayers, Ill.	1.43	1.39	-- 1906	--	200	.262	--	--	--	IWS
	do	do	do	--	--	Aug. 1958	52	150	.197	51.03	.69	766.87	
24-29a	Bodeman Reservoir	Trib. of West Creek	Brunswick, Ind.	2.73	2.39	Aug. 1947	--	52.1	*.036	--	--	--	SCS
	do	do	do	--	--	Sept. 1960	13	44.8	*.031	56.7	.23	284	
24-30	C B & Q R. R. Lake	Trib. of Sangamon River	Tallula, Ill.	.85	.84	-- 1902	--	31.7	.067	--	--	--	IWS
	do	do	do	--	--	July 1952	50	15.4	.033	49.3	.39	419	
24-31	Edwards Lake	Trib. of Cahokia Creek	Gillespie, Ill.	.42	.40	-- 1949	--	74.2	.351	--	--	--	IWS
	do	do	do	--	--	Aug. 1958	9	68.0	.322	46.86	1.72	1,755.47	
24-32	Michaeson's Lake	Trib. of Kaskaskia River	Vandalia, Ill.	.27	.26	-- 1943	--	19.7	.137	--	--	--	IWS
	do	do	do	--	--	Aug. 1958	15	16.0	.111	55.66	.93	1,127.33	
24-33	Glazier No. 1	Lost Creek	Elsberry, Mo.	1.05	1.04	Apr. 1956	--	1/72.5	--	--	--	--	SCS
	do	do	do	--	--	July 1960	4.3	70.3	--	--	.49	--	
24-34	I. C. at Kinmundy	Trib. of East Fork of Kaskaskia River	Kinmundy, Ill.	.618	.581	-- 1902	--	174.14	.459	--	--	--	IWS
	do	do	do	--	--	July 1959	57	149.10	.323	35.5	.756	984.5	
24-35	King's Lake	Trib. of Cahokia Creek	Eagarville, Ill.	.46	.44	-- 1921	--	158.6	.680	--	--	--	IWS
	do	do	do	--	--	June 1958	37	139.5	.598	54.62	1.17	1,306.73	
24-36	Lake Coulterville	South Fork Mud Creek	Coulterville, Ill.	1.22	1.18	-- 1940	--	200	.276	--	--	--	IWS
	do	do	do	--	--	Aug. 1954	14	188	.260	34	.73	541	
24-37	Lake Bunker Hill	Wood River	Bunker Hill, Ill.	7.19	7.15	-- 1937	--	133	.033	--	--	--	IWS
	do	do	do	--	--	June 1954	17	36	.009	79.2	.80	1,380	
24-38	Lake Gillespie	Dry Fork	Gillespie, Ill.	5.73	5.62	-- 1923	--	799	.272	--	--	--	IWS
	do	do	do	--	--	July 1954	32	696	.237	37.9	.57	470	
24-39	Lake Nashville	Nashville Creek	Nashville, Ill.	1.39	1.33	-- 1936	--	320	.376	--	--	--	IWS
	do	do	do	--	--	Aug. 1954	19	289	.339	35.3	1.19	913	
24-40	Lake Staunton	East Creek	Staunton, Ill.	3.68	3.54	-- 1926	--	1,268	.630	--	--	--	IWS
	do	do	do	--	--	July 1954	28	1,140	.575	28.1	1.09	667	
24-41	Macoupin County Lake Club	Trib. of Hurricane Creek	Carlinville, Ill.	.28	.26	-- 1904	--	160.5	1.131	--	--	--	IWS
	do	do	do	--	--	June 1958	54	135.8	.976	37.13	1.76	1,423.32	
24-42a	Mine Pond No. 4	Trib. of Cahokia Creek	Wilsonville, Ill.	5.29	5.23	-- 1916	--	296.1	.110	--	--	--	IWS
	do	do	do	--	--	July 1958	42	141.6	.053	65.34	.70	996	
24-43	New Mt. Olive	do	White City, Ill.	5.21	5.12	-- 1938	--	432.8	.164	--	--	--	IWS
	do	do	do	--	--	June 1958	20	335.3	.127	52.35	.94	1,071.79	
24-44a	Panama Lake	Trib. of Indian Creek	Panama, Ill.	.85	.84	-- 1928	--	177.8	.536	--	--	--	IWS
	do	do	do	--	--	Aug. 1958	30	151.9	.458	48.2	1.01	1,130	
24-45	Power Farms Ponds	Trib. of Sangamon River	Cantrall, Ill.	.668	.666	-- 1940	--	4.2	.011	--	--	--	IWS
	do	do	do	--	--	July 1952	12	1.3	.003	54.9	.36	430	
24-46	Raccoon Lake	Raccoon Creek	Centralia, Ill.	68.4	67.3	-- 1943	--	4,696.3	.152	--	--	--	IWS
	do	do	do	--	--	Sept. 1959	16	4,143.6	.140	39.7	.47	406	
24-47	Robinson Pond	Terre Creek	Jacksonville, Ill.	.314	.306	-- 1900	--	47.1	.277	--	--	--	IWS
	do	do	do	--	--	July 1952	52	35.6	.209	46.8	.70	713.51	
24-48	Salem City	Trib. of Crooked Creek	Salem, Ill.	4.02	3.90	-- 1912	--	597.86	.233	--	--	--	IWS
	do	do	do	--	--	Aug. 1960	48	530.88	.207	40.8	.36	320	
24-49a	Virginia	Job's Creek	Virginia, Ill.	.828	.799	-- 1933	--	154	.345	--	--	--	IWS
	do	do	do	--	--	-- 1950	17	116	.260	*40	2.70	2,352	
24-50	Walton Club Lake	Long Branch	Litchfield, Ill.	2.72	2.67	-- 1862	--	376.3	.272	--	--	--	IWS
	do	do	do	--	--	July 1959	97	187.2	.136	48.4	.73	769.5	
24-51	Lock and Dam 26	Mississippi River	Alton, Ill.	171,470	--	May 1938	--	395,000	--	--	--	--	CE
	do	do	do	--	--	May 1945	7.1	390,200	--	--	--	--	
	do	do	do	--	--	Dec. 1945	.6	--	--	--	--	--	
	do	do	do	--	--	May 1946	.4	381,200	--	--	--	--	
	do	do	do	--	--	Oct. 1947	1.4	408,200	--	--	--	--	
	do	do	do	--	--	Apr. 1959	11.5	343,900	--	--	--	--	
24-52a	Pool No. 22	do	Saverton, Mo.	137,500	--	July 1938	--	93,858	--	--	--	--	CE
	do	do	do	--	--	Dec. 1954	16.33	85,231	.0017	--	.006	--	
	do	do	do	--	--	Dec. 1969	15	2/102,362	.0018	--	--	--	

UPPER MISSISSIPPI RIVER BASIN (FAIRMONT TO LOUISIANA)
Iowa, Skunk, and Des Moines River Basins

25-1	Pool No. 19 (Lake Cooper, Keokuk Dam)	Mississippi River	Keokuk, Iowa	119,000	—	—	1913	—	479,550	—	—	—	—	CE
	do	do	do	—	—	June	1928	15	370,300	—	—	—	—	
	do	do	do	—	—	June	1938	10	337,000	—	—	—	—	
	do	do	do	—	—	June	1946	8	312,216	—	—	—	—	
25-2	Upper Pine	Pine Creek	Eldora, Iowa	13.9	13.8	May	1934	—	660	.145	—	—	—	SCS
	do	do	do	—	—	Sept.	1947	13.3	452	.099	*60	1.14	*1,490	
25-3	Lake Calhoun	Fitch Creek	Galva, Ill.	13.1	13.0	Sept.	1924	—	3/425	—	—	—	—	IWS
	do	do	do	—	—	Aug.	1936	11.9	273	—	—	.977	1,190	
	do	do	do	—	—	July	1947	11.0	112	—	56	1.12	1,370	
25-4	Carthage	Long Creek	Carthage, Ill.	2.94	2.88	Mar.	1926	—	406.3	—	—	—	—	IWS
	do	do	do	—	—	Aug.	1949	23.4	308.2	—	*50	1.45	1,580	
25-5	McCraney Creek New Desilting Basin	McCraney Creek	Kinderhook, Ill.	52.0	50.2	Dec.	1936	—	*2,400	—	—	—	—	SCS
	do	do	do	—	—	Dec.	1939	3	*1,770	—	*85	4.18	7,740	
	do	do	do	—	—	—	1942	2	*1,664	—	*85	1.06	1,960	
25-6	Pine Lake	Pine Creek	Eldora, Iowa	15.34	15.24	—	1924	—	738	*.148	—	—	—	SCS
	do	do	do	—	—	Winter	1932	8	552	*.111	*60	1.52	*1,990	
25-7	Hadley Creek Old Desilting Basin	Hadley Creek	New Canton, Ill.	77.0	4/72.7	—	1921	—	3,000	*.067	—	—	—	SCS
	do	do	do	—	—	—	1936	15	1,845	*.041	*85	1.06	1,960	
25-8	Hadley Creek New Desilting Basin	do	do	77.0	4/72.7	Dec.	1936	—	3,080	*.068	—	—	—	SCS
	do	do	do	—	—	Dec.	1939	3	2,556	*.057	*85	2.41	4,460	
25-9	Kiser Creek Desilting Basin	Kiser Creek	do	66.0	5/59.6	Dec.	1936	—	*2,800	*.073	—	—	—	SCS
	do	do	do	—	—	Dec.	1939	3	*2,290	*.059	*85	2.85	5,280	
25-10	Beeds Lake	Unnamed	Hampton, Iowa	31.8	31.6	—	1935	—	1,154	*.069	—	—	—	SCS
	do	do	do	—	—	—	1946	11	1,070	*.064	*65	.24	340	
25-11	C M St. P & P RR Res.	Trib. of Big Creek	Madrid, Iowa	2.54	2.52	—	1903	—	43	*.069	—	—	—	SCS
	do	do	do	—	—	—	1918	15	25	*.040	*70	.476	726	
25-12a	Fairfield No. 3	Crow Creek	Fairfield, Iowa	2.05	2.00	—	1927	—	207	*.303	—	—	—	SCS
	do	do	do	—	—	—	1934	7	166	*.242	*51.6	2.96	3,327	
	do	do	do	—	—	July	1953	19	135	*.198	51.6	.795	893	
25-13	Springbrook	Spring Brook	Guthrie Center, Iowa	2.1	2.1	—	1936	—	185	*.386	—	—	—	SCS
	do	do	do	—	—	—	1946	10	172	*.359	*50	.638	695	
25-14	Pool No. 16	Mississippi River	Muscatine, Iowa	99,400	—	Mar.	1938	—	113,370	—	—	—	—	CE
	do	do	do	—	—	Nov.	1949	11.7	106,347	—	—	—	—	
25-15	Pool No. 20	do	Canton, Mo.	134,300	—	Oct.	1937	—	89,850	—	—	—	—	CE
	do	do	do	—	—	Nov.	1950	13.1	87,740	—	—	—	—	
25-16	Bloomfield	Unnamed	Bloomfield, Iowa	2.25	2.13	Sept.	1937	—	896	*.664	—	—	—	SCS
	do	do	do	—	—	Sept.	1951	14.0	831	*.616	*60	2.16	2,823	
25-17	Pool No. 17	Mississippi River	New Boston, Ill.	99,600	—	Aug.	1938	—	70,800	—	—	—	—	CE
	do	do	do	—	—	Mar.	1954	15.6	69,350	—	—	.001	—	
25-18	Pool No. 21	do	Quincy, Ill.	135,000	—	July	1938	—	78,040	—	—	—	—	CE
	do	do	do	—	—	Mar.	1954	15.7	69,570	—	—	.004	—	
25-19a	Coralville	Iowa River	Iowa City, Iowa	3,115	3,076	Sept.	1958	—	492,000	.47	—	—	—	CE
	do	do	do	—	—	Jan.	1964	5.33	485,400	.46	40	.402	350.2	
	do	do	do	—	—	Apr.	1968	4.33	480,110	.46	45	.397	443.2	

UPPER MISSISSIPPI RIVER BASIN (PRAIRIE DU CHIEN TO ROCK ISLAND) AND LAKE MICHIGAN DRAINAGE
Rock and Wapsipinicon River Basins

26-1a	Pool No. 15	Mississippi River	Rock Island, Ill.	88,500	—	Mar.	1934	—	39,432	—	—	—	—	CE
	do	do	do	—	—	Aug.	1938	4.5	38,546	—	—	.002	—	
	do	do	do	—	—	Nov.	1944	6.2	38,229	—	—	.001	—	
	do	do	do	—	—	Dec.	1946	2.1	39,224	—	—	-.005	—	
	do	do	do	—	—	Nov.	1948	1.9	37,643	—	—	.009	—	
	do	do	do	—	—	Nov.	1950	2.0	37,881	—	—	-.001	—	
	do	do	do	—	—	Nov.	1952	2.0	37,086	—	—	.004	—	
26-2	Backbone Lake (Forestville Lake)	Maquoketa River	Strawberry Pt., Iowa	116	116	July	1934	—	608	—	—	—	—	SCS
	do	do	do	—	—	Feb.	1942	7.6	539	—	—	.078	127	
	do	do	do	—	—	Feb.	1949	7.0	473	—	75.1	.082	134	
26-3	Pool No. 11	Mississippi River	Dubuque, Iowa	81,600	—	Apr.	1938	—	171,484	—	—	—	—	CE
	do	do	do	—	—	Feb.	1953	14.8	154,526	—	—	.014	—	

UPPER MISSISSIPPI RIVER BASIN (ST. PAUL TO PRAIRIE DU CHIEN)
Wisconsin, Root, Chippewa, and St. Croix River Basins

27-1	Elk Creek Lake	Elk Creek	Eau Claire, Wisc.	60	60	—	1926	—	683	*.018	—	—	—	SCS
	do	do	do	—	—	Oct.	1941	15	457	*.012	*70	.252	384	
27-2	Ettrick Mill Pond	N. Br. Beaver Creek	Ettrick, Wisc.	50.75	50.70	—	1871	—	127	—	—	—	—	SCS
	do	do	do	—	—	June	1939	68	46	—	*80	.023	40	
27-3	Marinuka (Davis Lake)	Beaver Creek	Galesville, Wisc.	138.6	138.2	—	1867	—	1,677	*.022	—	—	—	SCS
	do	do	do	—	—	June	1939	72	683	*.009	72.5	.100	158	
27-4	Prairie du Sac	Wisconsin River	Prairie du Sac, Wisc.	8,900	6/600	—	1914	—	100,000	—	—	—	—	CE
	do	do	do	—	—	—	1933	19	95,851	—	*90	.737	1,400	

1/ Conservation Pool.
2/ All 31 ranges show scour, partially due to removal of borrow material.
3/ Spillway raised 2.89 ft. in 1946. Original capacity was 285.6 ac.-ft. All sedimentation and storage loss data based on higher spillway elevation.

4/ Excludes 3.8 sq. mi. Mississippi River bottom land.
5/ Excludes 5.8 sq. mi. Mississippi River bottom land.
6/ Flow from 8,300 sq. mi. of drainage area passes through power dams, which act as silt traps.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
	UPPER MISSISSIPPI RIVER BASIN (ST. PAUL TO PRAIRIE DU CHIEN) WISCONSIN, ROOT, CHIPPEWA, AND ST. CROIX RIVER BASIN (Continued)												
27-5	Pool No. 5A	Mississippi River	Winona, Minn. - Wisc.	59,100	--	July 1936	--	30,000	--	--	--	--	CE
	do	do	do	--	--	Feb. 1946	9.6	28,690	--	*90	--	--	
27-4a	DA-1 - Fishbaugher	Trib. D. E. Willow Creek	Preston, Minn.	1.31	1.29	Nov. 1956	--	168.7	.474	--	--	--	SCS
	do	do	do	--	--	Mar. 1960	3.4	166.5	.468	*70	0.51	778	
	do	do	do	--	--	Apr. 1964	4.1	164.1	.461	*70	.43	656	
27-7	Franklin Farm Pond	Trib. of Spring Creek	Red Wing, Minn.	1.188	1.181	May 1958	--	110.5	*.345	--	--	--	SCS
	do	do	do	--	--	June 1962	4.1	104.56	*.327	*65	1.23	1,741	
27-8	Frieheit Farm Pond	Trib. of Zumbro River	Zumbro Falls, Minn.	.269	.268	Aug. 1955	--	14.27	*.166	--	--	--	SCS
	do	do	do	--	--	May 1962	6.8	13.29	*.155	49.5	.52	560	
27-9	Stelplaugh Farm Pond	Trib. of Corey Creek	Money Creek, Minn.	.231	.230	Sept. 1957	--	19.09	*.313	--	--	--	SCS
	do	do	do	--	--	May 1962	4.7	18.04	*.296	*65	.96	1,359	
27-10	Wahlfiel Farm Pond	Trib. of Zumbro River	Millville, Minn.	.386	.384	July 1955	--	27.98	*.227	--	--	--	SCS
	do	do	do	--	--	May 1962	6.8	26.71	*.217	*55	.49	587	
27-11	Wold Farm Pond	Trib. of Crooked Creek	Spring Grove, Minn.	.194	.193	Aug. 1954	--	10.50	*.157	--	--	--	SCS
	do	do	do	--	--	Jan. 1962	7.3	10.03	*.150	51.86	.331	374	
27-12	FD-28 Structure East Willow Creek Watershed	Trib. of Root River	Preston, Minn.	.436	.433	Oct. 1954	--	56.79	*.444	--	--	--	SCS
	do	do	do	--	--	June 1962	7.7	55.18	*.431	*65	.48	680	
27-13	E-3 Mensink	Trib. of E.E. Willow Creek	do	3.16	3.10	May 1956	--	292.2	.340	--	--	--	SCS
	do	do	do	--	--	Mar. 1960	3.83	291.0	.338	*50	.11	120	
	do	do	do	--	--	Apr. 1964	4.08	287.2	.334	*50	.30	330	
	UPPER MISSISSIPPI RIVER BASIN (ABOVE ST. PAUL)												
28-													
	LAKE SUPERIOR AND LAKE OF THE WOODS AREA (IN MINNESOTA)												
29-													
	RED RIVER OF THE NORTH BASIN												
30-1	Lake Bronson	Two Rivers	Bronson, Minn.	439	438.5	Summer 1940	--	3,792	--	--	--	--	SCS
	do	do	do	--	--	Oct. 1950	10	3,626	--	30.94	.038	25.6	
30-2	Frazee	Otter Tail River	Frazee, Minn.	210	24	-- 1926	--	155	--	--	--	--	SCS
	do	do	do	--	--	May 1952	26	137.1	--	20	.029	12.6	
30-3	Blabon Dam	Trib. of Goose River	Blabon, N. Dak.	1.219	1.188	-- 1935	--	130.69	*3.354	--	--	--	SCS
	do	do	do	--	--	Oct. 1955	20	107.29	*2.753	41	.98	875	
30-4	Dougherty Dam	N. Br. Forest River	Adams, N. Dak.	20.2	14.6	-- 1935	--	190.7	*.209	--	--	--	SCS
	do	do	do	--	--	Aug. 1957	22	139.6	*.194	25	.034	18.81	
30-5	Gustafson Farm Pond	Park River	do	3.755	3.670	-- 1941	--	38.70	*.284	--	--	--	SCS
	do	do	do	--	--	May 1955	14	33.36	*.245	*35	.101	76.99	
30-6	Gwinner Dam	Trib. of Wild Rice River	Gwinner, N. Dak.	.226	.222	-- 1936	--	12.92	*2.494	--	--	--	SCS
	do	do	do	--	--	Aug. 1956	20	10.29	*1.986	29	.59	372.6	
30-7	Madsen Farm Pond	Swan Creek	Wheatland, N. Dak.	27.76	27.76	-- 1939	--	22.82	*.026	--	--	--	SCS
	do	do	do	--	--	June 1956	17	19.10	*.022	*35	.0078	5.95	
30-8	Magnolia Dam	Buffalo Creek	do	12.16	11.72	-- 1908	--	157.92	*.406	--	--	--	SCS
	do	do	do	--	--	June 1956	48	127.40	*.328	*35	.052	39.64	
30-9	Mason Siding Dam	Rush River	Erie, N. Dak.	30.2	30.2	-- 1912	--	43.15	*.045	--	--	--	SCS
	do	do	do	--	--	June 1956	44	35.40	*.037	*35	.0058	4.44	
30-10	Malvin Bollerud Farm Pond	Park River	Adams, N. Dak.	1.125	1.121	-- 1942	--	16.78	*.412	--	--	--	SCS
	do	do	do	--	--	May 1955	13	14.47	*.355	*35	.1579	114.68	
30-11	Milton WPA Dam	do	Milton, N. Dak.	14.375	14.356	-- 1934	--	66.99	*.128	--	--	--	SCS
	do	do	do	--	--	May 1955	21	46.19	*.089	*35	.0676	51.93	
30-12	Norby Dam	Trib. of Wild Rice River	Havana, N. Dak.	.183	.183	-- 1934	--	5.227	*1.191	--	--	--	SCS
	do	do	do	--	--	Aug. 1956	22	4.467	*1.018	*45	.188	184.3	
30-13	Raleigh Dam	Dog Tooth Creek	Raleigh, N. Dak.	4.45	4.45	-- 1909	--	175.47	*.925	--	--	--	SCS
	do	do	do	--	--	July 1956	47	145.23	*.766	*35	.144	109.8	
30-14	Sioux Railroad Reservoir	Park River	Adams, N. Dak.	16.855	16.66	-- 1911	--	88.37	*.145	--	--	--	SCS
	do	do	do	--	--	May 1955	44	69.16	*.113	*35	.026	18.88	
30-15	White Lake	Trib. of Wild Rice River	Havana, N. Dak.	37.0	21.08	-- 1938	--	1,895.6	--	--	--	--	SCS
	do	do	do	--	--	-- 1957	19	1,808.8	--	32	.2167	151	
30-16	Baldhill Dam (Lake Ashtabula)	Sheyenne River	Valley City, N. Dak.	1/4,138	2/1,979	Dec. 1952	--	76,400	--	--	--	--	CE
	do	do	do	--	--	Jan. 1958	5.1	75,410	--	*90	.0981	192	
30-17	Homme Dam (Park River)	S. Br. Park River	Park River, N. Dak.	229	229	Jan. 1953	--	3,690	.204	--	--	--	CE
	do	do	do	--	--	Mar. 1958	5.2	3,382	.189	*90	.225	441	
	MISSOURI RIVER BASIN (NEBRASKA CITY TO HERMANN)												
31-1	Lake of the Ozarks (Bagnell Dam)	Osage River	Eldon, Mo.	14,000	13,900	Feb. 1931	--	2,087,223	.299	--	--	--	CE
	do	do	do	--	--	Oct. 1948	17.8	1,972,531	.283	59.2	.464	598	
31-2	Carl Chinquist	Trib. of West Nodaway River	Stanton, Iowa	.166	.163	June 1938	--	3/9.46	*.231	--	--	--	SCS
	do	do	do	--	--	May 1949	10.9	2.59	*.063	49.9	3.87	4,200	

31-3	L. R. Pedling	Trib. of Tarkio Creek	Westboro, Mo.	1.05	1.04	July	1939	—	3/31.80	*.132	—	—	—	SCS
	do	do	do	—	—	May	1949	9.8	1.93	*.008	62.5	2.93	3,900	
31-4	McDaniel Lake	Little Sac River	Springfield, Mo.	41.9	41.5	—	1929	—	3,452	—	—	—	—	SCS
	do	do	do	—	—	June	1940	11	3,207	—	*60	.538	703	
31-5	Doniphan Co. Old Desilting Basin	Chase Creek	Blair, Kans.	1.62	1.58	—	1920	—	199.2	*.324	—	—	—	SCS
	do	do	do	—	—	June	1937	4/10.0	23.6	*.048	84.4	8.58	15,770	
31-6	Leavenworth Co. State Lake	Unnamed Trib.	Tonganoxie, Kans.	3.83	3.56	Sept.	1931	—	3,916	3.701	—	—	—	SCS
	do	do	do	—	—	Nov.	1947	16.1	3,771	3.564	61.45	2.53	3,386	
31-7	Lyon Co. State (Reading Lake)	do	Reading, Kans.	2.19	5/1.79	—	1935	—	1,889	*3.428	—	—	—	SCS
	do	do	do	—	—	Aug.	1939	4.0	1,870	*3.394	*60	2.73	3,568	
31-8	Martin Farm Lake	Osage River	Ottawa, Kans.	.18	.17	Nov.	1930	—	33	*.599	—	—	—	SCS
	do	do	do	—	—	Oct.	1937	6.9	30	*.508	*60	2.82	3,685	
31-9b	Mission Lake	Mission Creek	Horton, Kans.	8.15	7.76	May	1924	—	1,866	*1.025	—	—	—	ABS
	do	do	do	—	—	Apr.	1937	13	1,563	*.858	62.2	3.00	4,064	
	do	do	do	—	—	July	1954	17.2	1,181	*.649	53.3	2.86	3,326	
	do	do	do	—	—	Aug.	1967	13.1	1,091.5	*.599	58.5	.88	12.65	
31-10	Moran	S. Fk. Osage River	Moran, Kans.	5.35	5.28	Apr.	1913	—	365	*.130	—	—	—	SCS
	do	do	do	—	—	Sept.	1939	26.4	298	*.106	49.15	.477	511	
31-11	Lake Olathe	Cedar Creek	Olathe, Kans.	6.20	6.11	July	1932	—	721	—	—	—	—	SCS
	do	do	do	—	—	June	1937	4.9	666	—	60.4	1.84	2,421	
	do	do	do	—	—	July	1949	12.1	488	—	57.6	2.41	3,023	
31-12	Richmond	Unnamed Trib.	Richmond, Kans.	2.17	2.07	—	1909	—	405	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1937	28	289	—	*60	2.00	2,614	
31-13	Lampe Farm Pond	Trib. of Muddy Creek	Howe, Nebr.	.065	.064	Aug.	1962	—	3.00	*.231	—	—	—	SCS
	do	do	do	—	—	Dec.	1968	6.3	2.04	*.157	57.4	2.41	3,010	
31-14	E. W. Newell	Trib. of Tarkio Creek	Tarkio, Mo.	.509	.505	July	1934	—	3/33.7	*.303	—	—	—	SCS
	do	do	do	—	—	May	1949	6/6.9	.03	*.0003	72.5	9.66	15,250	
31-15	Allerton	South Chariton River	Allerton, Iowa	4.98	4.82	—	1913	—	511	—	—	—	—	SCS
	do	do	do	—	—	Nov.	1939	25.5	401	—	*60	.896	1,170	
31-16	Centerville No. 2	Manson's Branch	Centerville, Iowa	2.70	2.68	—	1926	—	1,111	—	—	—	—	SCS
	do	do	do	—	—	—	1937	11	1,050	—	*60	2.12	2,760	
31-17	Lake of Three Fires	E. Fk. 102 River	Bedford, Iowa	6.15	5.97	July	1936	—	1,231	—	—	—	—	SCS
	do	do	do	—	—	Feb.	1950	13.6	860	—	—	4.58	—	
31-18	Dominey Reservoir	Longs Creek	Johnson, Nebr.	.703	.699	—	1952	—	13.4	—	—	—	—	SCS
	do	do	do	—	—	—	1956	4	10.7	—	*65	.964	1,365	
31-19	Hadley Reservoir	Trib. of Nemaha River	Brock, Nebr.	.12	.115	—	1947	—	11.37	—	—	—	—	SCS
	do	do	do	—	—	—	1956	9	10.35	—	*65	.962	1,334	
31-20a	Schneider Reservoir	Long Branch	Johnson, Nebr.	.188	.184	—	1952	—	10.4	—	—	—	—	SCS
	do	do	do	—	—	—	1956	4	6.23	—	*65	5.65	8,000	
31-21	Cox	Unnamed Trib.	Blue Rapids, Kans.	.31	.30	June	1946	—	6.92	—	—	—	—	SCS
	do	do	do	—	—	Mar.	1955	8.8	4.05	—	67.22	1.10	1,610	
31-22	Droege	do	Osage City, Kans.	.4	.4	Mar.	1944	—	3/12.89	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1955	11.5	8.4	—	60	.97	1,267	
31-23	Flinner	do	Leavenworth, Kans.	.64	.63	Nov.	1936	—	81.6	—	—	—	—	SCS
	do	do	do	—	—	Aug.	1957	20.7	26.1	—	*60	4.22	5,515	
31-24	Honey Creek No. A-2	Trib. of Honey Creek	Russell, Iowa	.278	.274	Oct.	1956	—	52.7	*.750	—	—	—	SCS
	do	do	do	—	—	Oct.	1958	2.00	52.3	*.744	66	.71	1,000	
	do	do	do	—	—	Sept.	1959	.92	52.2	*.742	66	.76	1,100	
	do	do	do	—	—	Jan.	1961	1.33	52.2	*.742	66	0	0	
31-25	Honey Creek No. A-4	do	do	.355	.346	Aug.	1957	—	64.9	*.724	—	—	—	SCS
	do	do	do	—	—	Oct.	1958	1.17	63.5	*.708	66	3.52	5,100	
	do	do	do	—	—	Aug.	1959	.83	63.3	*.706	66	.71	1,000	
	do	do	do	—	—	Jan.	1961	1.42	63.3	*.706	66	0	0	
31-26	Honey Creek No. F-1	do	do	1.213	1.200	Nov.	1955	—	184	*.600	—	—	—	SCS
	do	do	do	—	—	Oct.	1958	2.92	181	*.590	66	.77	1,100	
	do	do	do	—	—	Oct.	1959	1.00	179	*.584	66	2.02	2,900	
	do	do	do	—	—	Jan.	1961	1.25	179	*.584	66	0	0	
31-27	Honey Creek No. E-1	Trib. of Honey Creek	Russell, Iowa	.478	.470	July	1955	—	142	*1.175	—	—	—	SCS
	do	do	do	—	—	June	1958	2.92	140	*1.158	66	1.35	1,900	
	do	do	do	—	—	Sept.	1959	1.25	138	*1.142	66	2.61	3,800	
	do	do	do	—	—	Jan.	1961	1.33	138	*1.142	66	0	0	
31-28	Honey Creek No. I-1	do	do	.503	.495	Nov.	1955	—	106	*.833	—	—	—	SCS
	do	do	do	—	—	Oct.	1958	2.92	105	*.826	66	.85	1,200	
	do	do	do	—	—	Sept.	1959	.92	104	*.818	66	2.22	3,200	
	do	do	do	—	—	Jan.	1961	1.33	104	*.818	66	0	0	
31-29	Miller Ranch	Unnamed Trib.	Miller, Kans.	.71	.70	Nov.	1936	—	52.72	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1955	18.8	37.83	—	52.7	1.13	1,297	
31-30	Nolte	Trib. of Wolf River	Hiawatha, Kans.	.42	.42	May	1951	—	3/7.55	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1955	4.0	5.67	—	68.43	1.12	1,669	
31-31	Prairie Lake	Unnamed Trib.	Holton, Kans.	1.49	1.40	Oct.	1947	—	721.7	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1955	8.0	653.1	—	54.3	6.12	7,197	
31-32	Nyhart	do	Effingham, Kans.	.16	.15	Oct.	1949	—	3/49.4	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1957	7.9	46.3	—	*60	2.4	3,136	
31-33	Sabetha Lake	Deer Creek Trib. of Nemaha	Sabetha, Kans.	9.17	8.99	Jan.	1936	—	1,934	.804	—	—	—	ABS
	do	do	do	—	—	June	1951	15.5	1,310	.544	54.7	4.48	5,337	
	do	do	do	—	—	Aug.	1952	1.2	1,234	.513	53.7	7.06	6,980	
	do	do	do	—	—	Oct.	1953	1.1	1,259	.523	58.0	7/-2.52	—	
	do	do	do	—	—	Apr.	1955	1.5	1,206	.501	54.1	3.93	379	
	do	do	do	—	—	Apr.	1956	1.0	1,205	.501	55.0	.11	1,720	
	do	do	do	—	—	Apr.	1958	2.0	1,200	.499	56.2	.28	1,400	
	do	do	do	—	—	July	1960	2.3	1,117	.464	54.6	4.02	3,537	

1/ Total drainage area (exclusive of closed Devils Lake Basin with drainage area of 3,940 sq. mi.).
2/ About 2,150 sq. mi. of the total drainage area shown is also noncontributing.
3/ Water supply pool capacity. Reservoir has greater capacity at spillway crest elevation.
4/ Water diverted from basin after June, 1930.
5/ Excluding area of lake and 0.31 sq. mi. above stock pond.
6/ Reservoir silted full Dec. 1936; Crest raised June 1937; silted full again Dec. 1940; crest raised again May 1945; silted full again May 1949. Age computed as 6.9 yr.
7/ Minus (-) indicated scour or compaction (treated as negative sediment).
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE -FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
			MISSOURI RIVER BASIN (NEBRASKA CITY TO HERMAN) (Continued)										
31-34	Sutter	Unnamed Trib.	Effingham, Kans.	.17	.16	Jan. 1950	--	1/53.2	—	—	—	--	SCS
	do	do	do	--	—	Oct. 1957	7.7	50.86	—	*60	1.88	2,457	
31-35	Shirley Pond	do	Lawrence, Kans.	.18	.18	Mar. 1939	—	20.86	—	—	—	--	SCS
	do	do	do	--	—	Nov. 1956	17.7	19.05	—	86.7	.56	1,057.5	
31-36	Trueman	do	Fairview, Kans.	.18	.18	May 1947	—	1/3.78	—	—	—	--	SCS
	do	do	do	--	—	Oct. 1955	8.4	2.42	—	62.02	.90	1,201	
31-37	Thorne	Unnamed	Effingham, Kans.	1.17	1.16	Sept. 1946	--	1/29.13	—	—	—	--	SCS
	do	do	do	--	--	Aug. 1957	11	17.7	—	*60	.90	1,176	
31-38b	Ashland Reservoir	Brushy Creek	Ashland, Mo.	3.78	3.75	Apr. 1937	--	280.48	—	--	—	--	ARS
	do	do	do	—	—	Nov. 1949	12.6	212.08	—	69	1.44	2,178	
	do	do	do	--	--	Apr. 1951	1.4	208.14	—	68	.73	807	
	do	do	do	--	—	July 1955	4.3	202.23	—	*68	.36	543	
	do	do	do	--	—	June 1962	6.9	186.07	—	69.4	.62	1,038	
	do	do	do	—	--	July 1968	6.1	171.52	—	64.7	.62	614	
31-39	Higginsville Old City Lake	Trib. of Davis Creek	Higginsville, Mo.	2.728	2.633	Sept. 1924	—	320.0	—	—	—	—	ARS
	do	do	do	—	—	July 1964	39.8	304.0	—	55.94	2.061	2,511	
31-40	Beecher	Trib. of Brush Creek	do	.334	2/.181	Oct. 1950	—	32.26	—	—	—	—	ARS
	do	do	do	--	—	Aug. 1964	13.8	25.55	—	58.74	2.686	3,436	
31-41	Mitchell	Trib. of Washington Creek	Lawrence, Kans.	.119	.117	Oct. 1949	—	7.94	—	—	—	--	SCS
	do	do	do	--	—	June 1962	12.67	6.73	—	56.8	.85	1,051	
31-42	McGee	do	Clinton, Kans.	.61	.58	Jan. 1950	—	6.77	—	—	—	—	SCS
	do	do	do	—	—	May 1962	12.4	6.26	—	47.9	.07	73	
31-43	Kennedy	Trib. of Wakarusa River	Richland, Kans.	.23	.22	Apr. 1937	—	61.19	—	—	—	—	SCS
	do	do	do	—	—	May 1962	25.1	54.27	—	46.7	1.27	1,296	
31-44	Jertzenberger	do	Eudora, Kans.	.125	.122	May 1955	—	1/7.52	—	—	—	—	SCS
	do	do	do	—	—	June 1962	7.1	5.68	—	56.2	2.13	2,609	
31-45	Lake Elbo	Trib. of Kansas River	Manhattan, Kans.	3.20	3.14	Apr. 1949	—	206	.303	—	—	--	SCS
	do	do	do	—	—	Sept. 1965	16.33	176	.299	75	.58	947	
31-46	Rokey	Trib. of Nemaha River	Sabetha, Kansas	.284	.280	May 1956	—	1/13.97	—	—	—	—	SCS
	do	do	do	—	—	Apr. 1962	5.9	11.32	—	50.7	1.61	1,777	
31-47	Callahan Cr. Watershed (C-1)	Barclay Branch	Columbia, Mo.	5.625	5.515	Apr. 1967	—	1,017.07	—	—	—	—	ARS
	do	do	do	—	—	Aug. 1967	.25	1,010.8	—	61.6	4.57	4,144	
	do	do	do	—	—	Aug. 1969	2	989.3	—	53	1.94	2,379	
	do	do	do	—	—	July 1971	1.9	974.9	—	58.2	1.38	2,099	
31-48	Wakefield	Trib. Marias Des Cygnes	Mound City, Kans.	.83	.82	— 1936	—	19.67	.05	—	—	--	SCS
	do	do	do	--	—	July 1967	31	11.06	.03	*60	.34	444	
31-49	Paul	Trib. Elk Creek	Circleville, Kans.	.16	.16	Oct. 1958	—	18.07	.39	—	—	--	SCS
	do	do	do	—	—	July 1968	9.75	16.18	.35	*60	1.19	1,555	
31-50	Richter	Trib. Straight Creek	Holton, Kans.	.28	.27	Fall 1957	—	64.36	.80	--	--	—	SCS
	do	do	do	—	--	July 1968	11	59.98	.74	*60	1.47	1,921	
31-51	Renner	do	do	.23	.23	July 1956	—	31.26	.47	—	—	--	SCS
	do	do	do	—	—	July 1968	12	27.3	.41	*60	1.43	1,869	
31-52	Rogers	Big Sugar	Garnett, Kans.	1.17	1.16	— 1936	—	22.8	.049	—	—	—	SCS
	do	do	do	—	—	June 1967	31	14.1	.03	57	.24	350	
31-53	Lemon	Trib. of Elm Creek	Parker, Kans.	.19	.19	June 1961	—	6.7	.08	--	—	--	SCS
	do	do	do	—	—	July 1967	6	6.16	.08	60	.47	614	
31-54	Elm Common Reservoir	Trib. Bull Creek	Gardner, Kans.	.77	.69	July 1960	—	204.3	.607	—	—	—	SCS
	do	do	do	--	—	Sept. 1970	10.2	191.1	.568	*60	1.88	2,457	
31-55	Oberholtzer Pond	do	Edgerton, Kans.	.57	3/.50	Sept. 1960	—	14.883	.061	—	—	—	SCS
	do	do	do	--	—	Sept. 1970	10	11.03	.045	*65	.77	1,090	
31-56	Richmond Lake	Dry Branch	Richmond, Kans.	.86	.84	— 1955	—	232.86	.68	—	--	--	SCS
	do	do	do	—	—	July 1967	12	222.06	.65	*60	1.07	1,398	
31-57	Hall Pond	Trib. Little Sugar	Farlinville, Kans.	.38	.37	Feb. 1955	—	46.22	.25	--	—	—	SCS
	do	do	do	—	—	July 1967	12	42.07	.24	*60	.76	993	
31-58	Johnson Pond	do	Blue Mound, Kans.	.94	.93	— 1936	—	24.03	.05	—	—	--	SCS
	do	do	do	—	—	July 1967	31	17.3	.039	*60	.24	313	
			SMOKY HILL AND LOWER REPUBLICAN RIVER BASINS										
33-1	Ottawa County State Lake	Sand Creek	Bennington, Kans.	20.47	20.26	— 1929	—	1,001	—	—	—	—	SCS
	do	do	do	—	—	Apr. 1937	8.0	930	—	—	.438	—	
33-2	Sheridan County State Lake	Saline River	Quinter, Kans.	493	463	Aug. 1937	—	777	—	--	—	—	BR
	do	do	do	—	—	Aug. 1948	10.8	436	—	66.5	.0481	98.6	
33-3a	Kanopolis	Smoky Hill River	Kanopolis, Kans.	7,860	1/2,560	July 1946	—	446,000	—	--	—	--	CE
	do	do	do	—	—	Sept. 1960	14.2	436,320	—	*50.0	.27	294	
33-4	Allington Reservoir	Sicily Creek	Beatrice, Nebr.	.52	.512	— 1936	—	33.104	—	—	--	—	SCS
	do	do	do	—	—	— 1956	20	14.704	—	*65	1.77	2,506	
33-5	Harvard Farm Reservoir	Indian Creek	do	.675	.661	Aug. 1947	—	30.5	—	--	--	—	SCS
	do	do	do	--	—	May 1956	8.75	22.22	—	*65	1.600	1,982	
33-6	Beers Reservoir	Trib. of Little Blue River	Hubbell, Nebr.	.175	.171	— 1937	—	8.098	—	—	—	—	SCS
	do	do	do	—	—	— 1958	21	2.301	—	*65	1.566	2,217	

32-7	Fling Estate	do	do	.108	.103	—	1945	—	18.686	—	—	—	—	SCS
	do	do	do	—	—	—	1958	13	13.852	—	*65	3.44	4,870	
32-8	Hansmire Reservoir	do	Reynolds, Nebr.	—	—	—	1948	—	5.943	—	—	—	—	SCS
	do	do	do	—	—	—	1958	10	3.641	—	*65	2.56	3,634	
32-9	Kegbe Reservoir	do	Gladstone, Nebr.	.306	.302	—	1913	—	9.806	—	—	—	—	SCS
	do	do	do	—	—	—	1958	45	5.179	—	*65	.337	477	
32-10	Rehme Reservoir No. 1	Bloody Run	Virginia, Nebr.	.422	.404	—	1899	—	70.7	—	—	—	—	SCS
	do	do	do	—	—	—	1956	57	30.8	—	*70	1.66	2,531	
32-11	Rehme Reservoir No. 2	do	do	.102	.099	—	1946	—	9.67	—	—	—	—	SCS
	do	do	do	—	—	—	1956	10	8.06	—	*65	1.98	2,237	
32-12	Omer Reservoir	Trib. of Little Blue River	Gladstone, Nebr.	.292	.283	—	1946	—	5.587	—	—	—	—	SCS
	do	do	do	—	—	—	1958	12	3.735	—	*65	.587	766	
32-13	Feltheum Reservoir	do	Hubbell, Nebr.	.216	.212	—	1950	—	9.934	—	—	—	—	SCS
	do	do	do	—	—	—	1958	8	5.574	—	*65	2.523	3,572	
32-14	Ult Reservoir	do	do	.191	.187	—	1946	—	12.274	—	—	—	—	SCS
	do	do	do	—	—	—	1958	12	6.528	—	*65	2.508	3,551	
32-15a	Wendy Hereford Ranch	Unnamed Trib.	Lincoln, Kans.	1.40	1.19	Nov.	1936	—	41.6	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1955	19	23.6	—	*60	.79	1,032	
32-16a	Daugherty	do	Russell, Kans.	.97	.96	Sept.	1947	—	22.5	.435	—	—	—	SCS
	do	do	do	—	—	Sept.	1955	8	16.1	.311	*60	.83	1,085	
32-17	Lost Creek D-1	Trib. of Lost Creek	Lincoln, Kans.	1.56	1.52	Jan.	1957	—	1/ 92.43	—	—	—	—	SCS
	do	do	do	—	—	July	1962	5.5	86.67	—	47.1	.69	708	
32-18	Jepsen	Trib. of Spillman Creek	do	1.0	1.0	May	1953	—	1/ 47.73	—	—	—	—	SCS
	do	do	do	—	—	July	1962	9.2	39.44	—	61.2	.90	1,200	
32-19	Wilson Bros.	Trib. of Saline River	do	.22	.21	Sept.	1950	—	1/ 9.51	—	—	—	—	SCS
	do	do	do	—	—	July	1962	11.8	8.28	—	*60	.48	627	
32-20	[illegible]	do	do	.222	.220	Nov.	1953	—	1/ 16.65	—	—	—	—	SCS
	do	do	do	—	—	July	1962	8.75	15.23	—	70.6	.73	1,122	
32-21	Mastrup	Trib. of Timber Creek	Vliets, Kans.	.16	.15	July	1956	—	1/ 15.29	—	—	—	—	SCS
	do	do	do	—	—	May	1962	5.8	14.14	—	80.9	1.33	2,344	
32-22	Cornelison	Trib. of Black Vermillion	do	.125	.120	June	1956	—	1/ 8.16	—	—	—	—	SCS
	do	do	do	—	—	May	1962	5.9	5.84	—	63.6	3.25	4,500	
32-23	Bragg	Unnamed Trib.	Bern, Kans.	.35	.34	July	1958	—	1/ 27.30	—	—	—	—	SCS
	do	do	do	—	—	May	1962	3.8	25.45	—	99.4	1.44	1,863	
32-24	Osborne	Trib. of Holland Creek	Elmo, Kans.	.156	.152	June	1956	—	1/ 10.51	—	—	—	—	SCS
	do	do	do	—	—	June	1962	6	9.68	—	75.6	.92	1,515	
32-25	Jacobson	Trib. of Turkey Creek	Dayton, Kans.	.211	.206	July	1950	—	11.78	—	—	—	—	SCS
	do	do	do	—	—	June	1962	12	5.16	—	62.9	2.67	3,698	
32-26	Frey	do	Elmo, Kans.	.128	.127	Dec.	1954	—	1.78	—	—	—	—	SCS
	do	do	do	—	—	June	1962	7.6	1.47	—	55.7	.32	376	
32-27	Rush	Trib. of Smoky Hill River	Navarre, Kans.	.560	.544	Oct.	1954	—	4.88	—	—	—	—	SCS
	do	do	do	—	—	July	1962	7.75	2.71	—	53.4	.51	593	
32-28	Dodd	Trib. of Dutch Creek	Centralia, Kans.	.64	.63	Oct.	1953	—	1/ 74.07	—	—	—	—	SCS
	do	do	do	—	—	May	1962	8.6	65.34	—	53.2	1.62	1,878	
32-29	Harlan County	Republican River	Republican City, Nebr.	20,753	9,277	July	1951	—	890,000	—	—	—	—	CE
	do	do	do	—	—	Sept.	1962	11.25	837,900	—	72	.116	182	
32-30	Grant Tusler	Trib. of Saline River	Beverly, Kans.	.99	.96	Mar.	1952	—	1/ 71.9	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1965	13.6	60.9	—	*72	.84	1,317	
32-31	Holmes Pond	Trib. Elk Creek	Lenora, Kans.	1.84	1.83	Oct.	1957	—	32	.466	—	—	—	SCS
	do	do	do	—	—	Aug.	1970	12.8	24.99	.356	*75	.32	523	
32-32	Finney Pond	Trib. Third Creek	Simpson, Kans.	1.39	1.37	Oct.	1960	—	122.5	1.003	—	—	—	SCS
	do	do	do	—	—	June	1970	9.7	116	.951	*80	.33	923	
32-33	Chrisler Pond	Trib. Saline River	Victoria, Kans.	1.01	.26	Dec.	1956	—	99.66	1.055	—	—	—	SCS
	do	do	do	—	—	July	1970	13.6	56.93	1.02	*60	.77	1,004	
32-34	Cheney Pond	Trib. Dry Creek	Delphos, Kans.	1.08	.92	Dec.	1956	—	108.11	.939	—	—	—	SCS
	do	do	do	—	—	June	1970	13.5	99.11	.862	*60	.72	947	
32-35	Brown Pond	Trib. Little Blue	Washington, Kans.	1.20	1.16	Apr.	1956	—	86.033	.395	—	—	—	SCS
	do	do	do	—	—	June	1970	13.3	78.19	.36	*75	.51	831	
32-36	Berger Pond	Trib. Buffalo Creek	Randall, Kans.	.59	.57	June	1953	—	22.68	.38	—	—	—	SCS
	do	do	do	—	—	June	1970	17	11.51	.195	*65	1.15	1,632	
32-37	Newell Pond	Trib. Saline River	Bunker Hill, Kans.	1.77	1.41	Dec.	1960	—	104.86	.855	—	—	—	SCS
	do	do	do	—	—	July	1970	9.6	103.17	.846	*75	.12	204	
32-38	[illegible] Pond	Trib. of Wolf Creek	Prairie View, Kans.	1.31	1.1	Nov.	1954	—	100	1.68	—	—	—	SCS
	do	do	do	—	—	July	1970	15.7	91	1.54	*65	.52	733	
32-39	Jones Pond	Trib. Smoky Hill River	Arnold, Kans.	.93	.92	Dec.	1944	—	23.42	.556	—	—	—	SCS
	do	do	do	—	—	Aug.	1970	25.7	21.25	.506	*80	.09	158	
32-40	King Pond	Trib. Mortimer Creek	Glasco, Kans.	1.03	.57	Nov.	1960	—	47.3	.453	—	—	—	SCS
	do	do	do	—	—	June	1970	9.6	45.3	.431	*65	.36	507	
32-41	Kinney Pond	Trib. Buffalo Creek	Mankato, Kans.	.77	.71	Sept.	1950	—	91.9	1.279	—	—	—	SCS
	do	do	do	—	—	June	1970	19.8	88.4	1.196	*65	.25	348	
32-42	Lee Pond	Trib. Saline River	Tescott, Kans.	.85	.82	Apr.	1952	—	152.4	1.868	—	—	—	SCS
	do	do	do	—	—	June	1970	18.2	146.3	1.786	*55	.41	492	
32-43	Lindsley Pond	Trib. Mill Creek	Marysville, Kans.	.72	.65	Oct.	1960	—	39.2	.319	—	—	—	SCS
	do	do	do	—	—	June	1970	9.7	35.8	.291	*70	.53	811	
32-44	Lloyd Pond	Trib. Republican River	Scandia, Kans.	.85	.84	July	1932	—	23.15	.193	—	—	—	SCS
	do	do	do	—	—	July	1970	38	8.12	.078	*70	.38	575	
32-45	Lovell Pond	Beaver Creek	Franklin, Nebr.	2.16	2.08	July	1951	—	126.72	.937	—	—	—	SCS
	do	do	do	—	—	July	1970	19	95.86	.726	*65	.78	1,106	
32-46	Mallenkamp Pond	Trib. Saline River	Collyer, Kans.	6.15	6.12	Apr.	1961	—	90.6	.368	—	—	—	SCS
	do	do	do	—	—	Aug.	1970	9.3	79.4	.324	*65	.20	277	
32-47	North Pond	Trib. Smoky Hill River	McCracken, Kans.	2.43	.91	Mar.	1955	—	96.83	.651	—	—	—	SCS
	do	do	do	—	—	July	1970	15.3	91.55	.614	*65	.38	537	
32-48	Raleigh Pond	Trib. Salt Creek	Agenda, Kans.	.90	.80	Mar.	1964	—	81.5	.653	—	—	—	SCS
	do	do	do	—	—	July	1970	6.3	79.4	.635	*60	.42	544	

1/ Water supply pool capacity. Reservoir has greater capacity at spillway crest elevation.
2/ Excludes conservation pool area and terraced area of watershed.
3/ 7,860 prior to closure of Cedar Bluff Dam Sept. 1950.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
SMOKY HILL AND LOWER REPUBLICAN RIVER BASINS (Continued)													
32-49	Sanger Pond	Trib. S. F. Solomon River	Norland, Kans.	2.53	2.32	Mar. 1954	—	332.12	3.519	—	—	—	SCS
	do	do	do	—	—	Aug. 1970	16.4	307.5	3.271	*65	0.65	917	
32-50	Shurias Pond	Trib. Rose Creek	Munden, Kans.	1.61	1.61	July 1954	—	12.896	.058	—	—	—	SCS
	do	do	do	—	—	July 1970	16	6.8	.030	*70	.24	361	
32-51	Schurle Pond	Trib. Mall Creek	Green, Kans.	.73	.67	Nov. 1960	—	127.1	1.088	—	—	—	SCS
	do	do	do	—	—	June 1970	9.6	123.3	1.053	*60	.59	767	
32-52	Wals Pond	Trib. Saline River	Ogallah, Kans.	3.93	3.8	Aug. 1958	—	69.32	.389	—	—	—	SCS
	do	do	do	—	—	Aug. 1970	12	63.64	.357	*60	.12	162	
32-53	Woolhoff Pond	Trib. Five Creek	Morganville, Kans.	.61	.59	June 1954	—	29.05	.331	—	—	—	SCS
	do	do	do	—	—	July 1970	16.1	24.52	.278	*65	.48	680	
32-54	Herington City Lake	Lyons Creek	Herington, Kans.	17.24	16.93	May 1922	—	2,180	.68	—	—	—	SCS
	do	do	do	—	—	May 1966	44	1,455	.65	*70	.96	1,463	
UPPER REPUBLICAN, NORTH PLATTE, RIVER BASINS (FT. LARAMIE TO NORTH PLATTE) AND SOUTH PLATTE RIVER BASIN (SUBLETTE TO NORTH PLATTE)													
33-1	Wellfleet	Medicine Creek	Wellfleet, Nebr.	1/15.00	14.89	Oct. 1931	—	519	—	—	—	—	SCS
	do	do	do	—	—	May 1937	5.6	464	—	*65	.66	936	
33-2	Lockhart Farm Pond	Trib. of N. Platte River	Gering, Nebr.	2/2.564	2.558	Aug. 1939	—	15.64	.229	—	—	—	SCS
	do	do	do	—	—	July 1952	12.9	8.83	.129	57.3	.206	257	
33-3a	Harry Strunk Lake (Medicine Creek Dam)	Medicine Creek	Cambridge, Nebr.	656	653	Aug. 1949	—	92,817	1.691	—	—	—	BR
	do	do	do	—	—	Oct. 1951	2.16	90,930	1.656	71.4	1.34	2,084	
	do	do	do	—	—	Dec. 1962	11.17	88,463	1.615	70.3	.31	475	
33-4a	Flanagin Reservoir	Trib. of Republican River	Bartley, Nebr.	.900	.497	— 1940	—	9.71	—	—	—	—	SCS
	do	do	do	—	—	— 1956	16	4.09	—	*70	.602	918	
33-5a	Gallatin Reservoir	Trib. of Beaver Creek	Danbury, Nebr.	3.44	3.09	Apr. 1946	—	419.5	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1956	10.4	381.6	—	*70	1.18	1,800	
33-6	Klein Reservoir	Trib. of Republican River	McCook, Nebr.	1.59	1.57	— 1936	—	106.2	—	—	—	—	SCS
	do	do	do	—	—	— 1956	20	56.0	—	*70	1.58	2,409	
33-7	Dempsey Pond	Trib. of East Curtis Creek	Moorefield, Nebr.	*.526	.51	June 1949	—	11.00	—	—	—	—	ARS
	do	do	do	—	—	July 1953	4	10.10	—	—	.447	—	
	do	do	do	—	—	June 1958	4.9	9.12	—	75.6	.392	445	
33-8a	Reichelt Stock Pond	Unnamed	Julesburg, Colo.	.72	.72	Oct. 1950	—	22.12	—	—	—	—	SCS
	do	do	do	—	—	Nov. 1957	7	20.69	—	80.4	.28	490	
33-9	Deshaka Pond	Trib. Beaver Creek	Atwood, Kans.	1.41	1.4	Nov. 1948	—	18.19	.968	—	—	—	SCS
	do	do	do	—	—	Aug. 1970	21.8	9.74	.541	*70	.28	422	
33-10	Frisbey Pond	Trib. Sappa Creek	Norcatur, Kans.	3.12	3.28	Dec. 1958	—	72.48	.683	—	—	—	SCS
	do	do	do	—	—	July 1970	11.6	63.17	.596	*60	.24	330	
33-11	Walker Pond	Trib. Beaver Creek	Atwood, Kans.	4.78	4.77	Oct. 1948	—	20.96	.329	—	—	—	SCS
	do	do	do	—	—	Aug. 1970	21.8	13.82	.215	*70	.07	105	
33-12	Stroud Pond	Trib. Jones Canyon Creek	McDonald, Kans.	.83	.82	Nov. 1958	—	111.25	10.053	—	—	—	SCS
	do	do	do	—	—	July 1970	11.7	102.01	9.274	*70	.96	1,467	
33-13	Franktown-Parker W-1 W'shed	Willow Creek	Castle Rock, Colo.	7.6	7.57	June 1963	—	387	*1.92	—	—	—	SCS
	do	do	do	—	—	Jan. 1967	4	337.4	*1.67	—	1.6	—	
33-14	Bondegard Stock Pond	Sedgwick Draw	Lodgepole, Nebr.	1.5	1.5	— 1952	—	27.68	1.20	—	—	—	SCS
	do	do	do	—	—	Dec. 1967	15	23.88	1.06	*75	.17	277	
NORTH PLATTE RIVER BASIN (ABOVE FT. LARAMIE) SOUTH PLATTE RIVER BASIN (ABOVE SUBLETTE)													
34-1	Castlewood	Cherry Creek	Denver, Colo.	167.2	166.9	— 1890	—	3,834	—	—	—	—	SCS
	do	do	do	—	—	3/Aug. 1933	43	3,126	—	77.5	.099	167	
34-2	Kenwood	do	do	387	386	Mar. 1936	—	9,802	—	—	—	—	SCS
	do	do	do	—	—	June 1938	2.25	9,710	—	75.6	.106	175	
	do	do	do	—	—	June 1939	1.00	9,424	—	—	.741	1,220	
34-3a	Guernsey Reservoir	North Platte River	Guernsey, Wy.	15,006	675	Feb. 1927	—	73,810	—	—	—	—	BR
	do	do	do	—	—	Jan. 1931	3.83	67,840	—	—	.28	—	
	do	do	do	—	—	Jan. 1933	2.00	65,090	—	—	.26	—	
	do	do	do	—	—	Jan. 1935	2.00	62,960	—	—	.20	—	
	do	do	do	—	—	Feb. 1937	2.10	60,930	—	—	.18	—	
	do	do	do	—	—	Feb. 1939	2.00	58,430	—	—	.23	—	
	do	do	do	—	—	Jan. 1941	2.00	56,600	—	—	.17	—	
	do	do	do	—	—	Jan. 1944	3.00	53,180	—	—	.21	—	
	do	do	do	—	—	July 1947	3.50	49,150	—	—	.21	343	
	do	do	do	—	—	June 1957	9.92	44,800	—	60.7	.103	107	
	do	do	do	—	—	Dec. 1966	9.40	4/45,220	.0467	56	—	—	
34-4a	Lake Cheesman	South Platte R. and Goose or Lost Park Cr.	Deckers, Colo.	1,766	1,460	Oct. 1900	—	79,064	.375	—	—	—	BR
	do	do	do	—	—	Sept. 1931	31	77,958	.386	*70	.025	37.2	

34-5	Pathfinder	North Platte River	Alcova, Wy.	10,700	5/3,315	June	1909	—	1,056,300	—	—	—	—	BR
	◆	◆	◆	—	—	July	1950	41.1	1,015,876	—	61.1	7/.070	7/130	
34-6	Seminoe	◆	Leo, Wy.	7,348	7,317	—	1939	—	1,020,000	1.158	—	—	—	BR
	◆	◆	◆	—	—	Sept.	1950	11.5	1,012,000	1.149	65	6/.108	6/253	
34-7	Englewood	Little Dry Creek	Littleton, Colo.	9.40	9.25	—	1936	—	1,282.3	—	—	—	—	SCS
	◆	◆	◆	—	—	Aug.	1956	19.5	1,215.2	—	—	.37	—	
34-8a	B-3 Kiowa Creek Watershed	Kiowa Creek	Elbert, Colo.	2.92	2.89	July	1956	—	147.99	*.27	—	—	—	SCS
	◆	◆	◆	—	—	Mar.	1966	10	138.22	*.25	80	.32	937	
34-9	Round Butte	Coal Creek	Wellington, Colo.	11.7	11.6	—	1905	—	831	—	—	—	—	SCS
	◆	◆	◆	—	—	June	1965	60	785	—	69	.07	105	
34-10a	Q-51 Kiowa Creek Watershed	Kiowa Creek	Elbert, Colo.	.56	.56	June	1957	—	32.30	*.31	—	—	—	SCS
	◆	◆	◆	—	—	Jan.	1966	9	30.17	*.29	80	.43	760	
34-11	J-33 Kiowa Creek Watershed	◆	Eastonville, Colo.	1.07	1.06	July	1956	—	42.55	—	—	—	—	SCS
	◆	◆	◆	—	—	July	1965	9	42.05	—	80	.03	67	
34-12	B-9 Kiowa Creek Watershed	◆	Elbert, Colo.	.65	.64	Dec.	1955	—	49.36	—	—	—	—	SCS
	◆	◆	◆	—	—	July	1965	9.5	48.82	—	80	.09	157	
34-13	Slab Canyon CCC	Slab Canyon	Wellington, Colo.	3.17	3.08	July	1937	—	311.4	*1.85	—	—	—	SCS
	◆	◆	◆	—	—	Feb.	1966	30	289.3	*1.72	—	.25	—	
				MISSOURI RIVER BASIN (ABOVE BLAIR TO NEBRASKA CITY) PLATTE RIVER BASIN (BELOW NORTH PLATTE)										
35-1	Otto Buck	Trib. of Soldier River	Ricketts, Iowa	.159	.157	Sept.	1944	—	9/6.91	*.177	—	—	—	SCS
	◆	◆	◆	—	—	Apr.	1949	4.6	2.28	*.058	36.9	6.41	7,090	
35-2	Fred Brown	Trib. of Willow River	Logan, Iowa	.100	.097	June	1941	—	9/11.09	*.596	—	—	—	SCS
	◆	◆	◆	—	—	May	1949	7.9	4.61	*.230	63.2	9.51	13,080	
35-3	William Schoch	Trib. of Elkhorn Creek	Elkhorn, Iowa	.208	.204	May	1940	—	9/12.60	*.238	—	—	—	SCS
	◆	◆	◆	—	—	May	1949	9.0	4.51	*.087	56.2	6.41	5,390	
35-4	G. & A. Evers Lower Reservoir	Trib. of Boyer River	Denison, Iowa	.187	.139	Dec.	1938	—	9/6.63	*.138	—	—	—	SCS
	◆	◆	◆	—	—	Apr.	1949	10.3	1.26	*.026	69.8	3.75	5,700	
35-5	G. & A. Evers Upper Reservoir	◆	◆	.045	.044	Mar.	1939	—	9/1.90	*.158	—	—	—	SCS
	◆	◆	◆	—	—	Apr.	1949	10.1	.22	*.018	71.4	3.77	5,870	
35-6	Charles Fiebold	◆	Dunlap, Iowa	.428	.425	May	1945	—	9/12.63	*.125	—	—	—	SCS
	◆	◆	◆	—	—	Apr.	1949	3.9	2.90	*.028	63.1	5.07	9,070	
35-7a	C. T. Good	Trib. of E. Nishnabotna R.	Stennett, Iowa	.081	.079	Dec.	1940	—	14.68	*.762	—	—	—	SCS
	◆	◆	◆	—	—	May	1949	8.4	12.82	*.675	63.9	2.51	3,490	
	◆	◆	◆	—	—	June	1952	3.1	11.98	*.631	57.9	3.43	4,330	
35-8	Otto Gosler	Trib. of Middle Soldier R.	Charter Oak, Iowa	.089	.086	May	1940	—	9/10.18	*.485	—	—	—	SCS
	◆	◆	◆	—	—	Mar.	1949	8.8	8.29	*.395	68.8	2.30	3,750	
35-9	Thomas Rodkin	Trib. of Boyer River	Dow City, Iowa	.130	.127	Aug.	1942	—	9/19.39	*.588	—	—	—	SCS
	◆	◆	◆	—	—	May	1949	10/2.8	.21	*.006	83.2	53.9	97,760	
35-10	Fred Hallrab	Trib. of Willow Creek	Ute, Iowa	.117	.113	Aug.	1944	—	9/16.93	*.365	—	—	—	SCS
	◆	◆	◆	—	—	Mar.	1949	4.6	12.76	*.265	50.3	6.35	8,050	
35-11b	Jones Creek	Jones Creek	Pisgah, Iowa	2.26	2.23	Feb.	1942	—	256.2	*.527	—	—	—	SCS
	◆	◆	◆	—	—	Jan.	1949	6.9	222.8	*.460	47.8	2.11	2,197	
	◆	◆	◆	—	—	Sept.	1950	1.7	211.5	*.439	48.8	2.60	3,620	
	◆	◆	◆	—	—	Jan.	1953	2.3	199.2	*.413	48.6	2.40	3,250	
35-12a	Sam La Frentz	Trib. of Boyer River	Denison, Iowa	.155	.152	May	1942	—	15.38	*.577	—	—	—	SCS
	◆	◆	◆	—	—	Apr.	1949	6.9	10.38	*.392	56.7	4.67	5,767	
	◆	◆	◆	—	—	July	1953	4.2	9.47	*.357	62.0	1.41	1,996	
35-13	Jensen-O'Neil Farm Pond	Trib. of L. Nemaha River	Syracuse, Nebr.	.199	.193	Nov.	1936	—	23.7	*.658	—	—	—	SCS
	◆	◆	◆	—	—	Nov.	1948	12.0	16.8	*.467	56.9	2.97	3,680	
35-14	Peterson Farm Pond	◆	◆	.075	.074	Oct.	1936	—	2.06	*.150	—	—	—	SCS
	◆	◆	◆	—	—	July	1948	11.75	.16	*.012	99.7	2.19	2,850	
35-15b	Alfred Lage	Trib. of Elk Creek	Aspinwall, Iowa	.184	.182	June	1941	—	11.23	*.229	—	—	—	SCS
	◆	◆	◆	—	—	Apr.	1949	7.8	8.56	*.175	53.2	1.88	2,180	
	◆	◆	◆	—	—	June	1952	3.2	7.14	*.146	51.9	2.12	2,735	
35-16a	Herman Lage	◆	◆	.038	.036	July	1941	—	12.07	*1.207	—	—	—	SCS
	◆	◆	◆	—	—	Apr.	1949	7.8	10.83	*1.083	58.8	4.437	5,270	
	◆	◆	◆	—	—	June	1952	3.2	8.98	*.898	68.5	16.1	17,350	
35-17a	Howard Hutton	Trib. of Boyer River	Denison, Iowa	.100	.098	July	1944	—	16.44	*.961	—	—	—	SCS
	◆	◆	◆	—	—	Apr.	1949	4.8	12.56	*.735	69.1	8.26	12,431	
	◆	◆	◆	—	—	July	1953	4.3	5.71	*.336	67.1	16.3	30,900	
35-18a	Wilbur Meyer	◆	◆	.297	.293	Nov.	1944	—	43.80	*.576	—	—	—	SCS
	◆	◆	◆	—	—	Apr.	1949	4.4	37.33	*.491	56.3	3.08	6,160	
	◆	◆	◆	—	—	June	1952	3.2	34.63	*.456	58.1	2.80	3,520	
35-19a	Max Miller No. 1	Trib. of W. Nishnabotna R.	Macedonia, Iowa	.223	.218	Nov.	1941	—	43.84	*.877	—	—	—	SCS
	◆	◆	◆	—	—	May	1949	7.5	35.46	*.709	65.2	5.12	7,280	
	◆	◆	◆	—	—	June	1952	3.1	32.10	*.642	69.7	4.95	7,520	
35-20b	Max Miller No. 5	◆	◆	.233	.230	Nov.	1941	—	53.19	*1.083	—	—	—	SCS
	◆	◆	◆	—	—	May	1949	7.5	41.96	*.837	72.6	6.98	10,405	
	◆	◆	◆	—	—	June	1952	3.1	41.06	*.790	76.5	1.23	1,996	
35-21a	Harvey Hunt	Trib. of Boyer River	Denison, Iowa	.336	.330	Oct.	1944	—	41.35	*.462	—	—	—	SCS
	◆	◆	◆	—	—	Apr.	1949	4.5	33.08	*.395	56.8	11/3.66	11/6,730	
	◆	◆	◆	—	—	June	1952	3.2	29.22	*.360	55.5	11/3.67	11/6,440	
35-22a	Tracy North	◆	Vail, Iowa	.245	.238	Nov.	1939	—	48.44	*1.168	—	—	—	SCS
	◆	◆	◆	—	—	Mar.	1949	9.4	40.04	*.956	52.7	3.64	6,410	
	◆	◆	◆	—	—	July	1953	4.3	37.41	*.893	55.7	2.97	3,180	
35-23	Lake Ericson	Cedar River	Ericson, Nebr.	12/900	41	May	1915	—	1,652	—	—	—	—	BR
	◆	◆	◆	—	—	Apr.	1948	32.9	1,066	—	80	.436	736	
35-24a	Roe Farm Pond	Trib. of Turkey Creek	Dannebrog, Nebr.	.18	.18	July	1947	—	*1.0	*.104	—	—	—	SCS
	◆	◆	◆	—	—	Aug.	1953	5	*.443	*.087	65	.30	592	

1/ Excludes 28 sq. mi. of noncontributing drainage.
2/ Drainage area includes partial sediment trap for 0.133 sq. mi. and is estimated to retain 0.5 ac.-ft. of sediment per year.
3/ Dam failed in 1933; survey made in 1939.
4/ Gain in capacity due to sediments sluiced from dam.
5/ Excludes 7,348 sq. mi. above Seminoe Dam, which was closed in April 1939.
6/ For period April 1939 - July 1950.
7/ For period June 1909 - July 1950.
8/ Below elevation 6,340 which is 17 ft. below spillway crest.
9/ Water supply pool capacity. Reservoir has greater capacity at spillway crest elevation.
10/ Reservoir silted full June 1944.
11/ Includes 0.13 ac.-ft. of sediment above emergency spillway.
12/ Area assumed to contribute surface runoff is 41 mi.²

SUMMARY OF

RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
			MISSOURI RIVER BASIN (ABOVE BLAIR TO NEBRASKA CITY) PLATTE RIVER BASIN (BELOW NORTH PLATTE) (Continued)										
35-25	Hose Farm Pond	Trib. of Dead Horse Creek	Loup City, Nebr.	1.80	1.79	Aug. 1948	—	15.6	*.163	—	—	—	SCS
	do	do	do	—	—	Aug. 1953	5	13.9	*.145	*65	.20	283	
35-26	Muller Farm Pond	Trib. of Oak Creek	Farwell, Nebr.	.358	.354	Aug. 1949	—	15.40	*.811	—	—	—	SCS
	do	do	do	—	—	Aug. 1953	4	14.39	*.757	*65	.71	1,005	
35-27	Rasicka Farm Pond	do	do	.27	.26	Aug. 1949	—	30.76	*2.135	—	—	—	SCS
	do	do	do	—	—	Aug. 1953	4	28.03	*1.947	*65	2.62	3,709	
35-28	Brehmer Reservoir	Trib. of L. Nemaha River	Dunbar, Nebr.	.184	.180	— 1952	—	11.78	—	—	—	—	SCS
	do	do	do	—	—	— 1957	5	9.93	—	*65	2.01	2,846	
35-29	Cook Reservoir	Trib. of Papillion Creek	Arlington, Nebr.	.391	.383	— 1936	—	29.4	—	—	—	—	SCS
	do	do	do	—	—	— 1957	21	21.1	—	*65	.921	1,304	
35-30a	Ingwerson Reservoir No. 1	Turkey Creek	Louisville, Nebr.	.211	.209	— 1952	—	12.17	—	—	—	—	SCS
	do	do	do	—	—	— 1957	5	10.19	—	*65	1.89	2,676	
35-31	Ingwerson Reservoir No. 2	do	do	.076	.073	— 1952	—	.72	—	—	—	—	SCS
	do	do	do	—	—	— 1957	5	.19	—	*65	1.43	2,024	
35-32	Ingwerson Reservoir No. 3	do	do	.335	.331	— 1954	—	18.73	—	—	—	—	SCS
	do	do	do	—	—	— 1957	3	17.01	—	*65	1.71	2,421	
35-33	Kruse Reservoir	Papillion Creek	Elkhorn, Nebr.	.80	.779	— 1936	—	61.5	—	—	—	—	SCS
	do	do	do	—	—	— 1957	21	11.2	—	*65	2.99	4,233	
35-34	Lego Reservoir	Unnamed	Douglas, Nebr.	.263	.257	— 1946	—	12.8	—	—	—	—	SCS
	do	do	do	—	—	— 1957	11	4.33	—	*65	2.93	4,148	
35-35	Niebuer Reservoir	Trib. of L. Nemaha River	Dunbar, Nebr.	.163	.158	— 1949	—	14.48	—	—	—	—	SCS
	do	do	do	—	—	— 1957	8	10.92	—	*65	2.72	3,851	
35-36	O'Brien Reservoir	South Cedar Creek	Manley, Nebr.	.417	.412	— 1937	—	22.09	—	—	—	—	SCS
	do	do	do	—	—	— 1957	20	7.28	—	*65	1.77	2,506	
35-37a	O'Neill Reservoir	Trib. of L. Nemah River	Syracuse, Nebr.	.140	.130	— 1936	—	35.69	—	—	—	—	SCS
	do	do	do	—	—	— 1957	21	23.98	—	*65	4.29	6,073	
35-38	Pickrell Reservoir No. 1	Russell Creek	Unadilla, Nebr.	.336	.326	— 1949	—	35.76	—	—	—	—	SCS
	do	do	do	—	—	— 1957	8	32.35	—	*65	1.27	1,798	
35-39	Pickrell Reservoir No. 2	do	do	.102	.100	— 1954	—	3.18	—	—	—	—	SCS
	do	do	do	—	—	— 1957	3	2.96	—	*65	.725	1,026	
35-40	Pickrell Reservoir No. 3	do	do	.088	.086	— 1939	—	5.04	—	—	—	—	SCS
	do	do	do	—	—	— 1957	18	1.60	—	*65	2.31	3,270	
35-41	Pickrell Reservoir No. 4	do	do	.114	.113	— 1934	—	2.30	—	—	—	—	SCS
	do	do	do	—	—	— 1957	23	1.60	—	*65	.272	385	
35-42	Pollard Reservoir	Weeping Water Creek	Nehawka, Nebr.	.100	.098	— 1936	—	7.63	—	—	—	—	SCS
	do	do	do	—	—	— 1957	21	4.38	—	*65	1.55	2,196	
35-43	Hose Farm Reservoir	N. Fork Little Fox River	Otoe, Nebr.	.077	.076	— 1948	—	8.12	—	—	—	—	SCS
	do	do	do	—	—	— 1957	9	6.71	—	*65	2.06	2,888	
35-44	Bender Reservoir	Long Creek	Fort Calhoun, Nebr.	.30	.298	— 1949	—	12.7	—	—	—	—	SCS
	do	do	do	—	—	— 1957	8	4.12	—	*65	3.57	5,054	
35-45b	Male Creek "A"	Trib. of Nishnabotna River	Malvern, Iowa	.188	.177	July 1954	—	43.9	*1.109	—	—	—	SCS
	do	do	do	—	—	Oct. 1956	2.33	42.0	*1.061	73.4	4.63	7,400	
	do	do	do	—	—	Sept. 1957	.92	40.2	*1.015	73.4	11.40	18,200	
	do	do	do	—	—	Oct. 1958	1.08	39.6	*1.000	72.3	3.11	4,900	
	do	do	do	—	—	Jan. 1960	1.33	38.9	*.982	58.7	2.99	3,800	
	do	do	do	—	—	Dec. 1960	.92	36.5	*.922	60.3	14.90	19,000	
	do	do	do	—	—	Feb. 1962	1/1.00	36.0	*.909	2/60.3	2.99	3,960	
	do	do	do	—	—	Jan. 1963	.92	34.7	*.876	69.5	7.85	11,800	
	do	do	do	—	—	Jan. 1964	1.00	3/35.56	*.897	2/60.3	4/-4.66	—	
	do	do	do	—	—	Jan. 1965	1.00	34.65	*.870	2/60.3	6.20	8,140	
	do	do	do	—	—	Jan. 1966	1.00	31.97	*.807	2/60.3	13.99	18,370	
	do	do	do	—	—	Jan. 1967	1.00	32.81	.828	2/60.3	4/-6.22	—	
	do	do	do	—	—	Jan. 1968	1.00	30.88	.780	2/60.3	10.91	14,328	
	do	do	do	—	—	Mar. 1969	1.17	31.37	.792	5/75.5	4/-2.372	—	
35-46b	Male Creek "B"	do	do	.383	.368	Aug. 1956	—	55.3	*.684	—	—	—	SCS
	do	do	do	—	—	Oct. 1956	2.17	52.6	*.651	72.6	3.31	5,200	
	do	do	do	—	—	Sept. 1957	.92	51.8	*.641	69.5	2.55	3,900	
	do	do	do	—	—	Oct. 1958	1.08	50.3	*.623	59.2	3.66	4,700	
	do	do	do	—	—	Jan. 1960	1.33	47.6	*.589	67.2	5.52	8,100	
	do	do	do	—	—	Dec. 1960	.92	47.1	*.583	55.3	1.55	1,900	
	do	do	do	—	—	Feb. 1962	1/1.08	45.1	*.558	2/55.3	5.11	6,150	
	do	do	do	—	—	Jan. 1963	.92	43.5	*.538	62.1	4.73	6,400	
	do	do	do	—	—	Jan. 1964	1.00	44.6	*.552	51.8	4/-2.88	—	
	do	do	do	—	—	Jan. 1965	1.00	42.7	*.528	51.8	4.92	5,550	
	do	do	do	—	—	Jan. 1966	1.00	36.5	*.452	51.8	16.91	19,080	
	do	do	do	—	—	Aug. 1966	.58	39.89	.494	51.8	4/-15.706	—	
	do	do	do	—	—	June 1969	2.75	40.069	.50	5/71.64	4/-.298	—	
35-47b	Male Creek "C"	do	do	6/.916	.333	July 1954	—	76.2	—	—	—	—	SCS
	do	do	do	—	—	Oct. 1956	2.33	73.8	—	53.9	3.28	3,900	
	do	do	do	—	—	Sept. 1957	.92	69.1	—	66.1	15.50	22,300	

	do	do	do	--	--	Oct.	1958	1.08	67.8	--	59.8	3.90	5,100	
	do	do	do	--	--	Jan.	1960	1.33	66.9	--	61.9	1.98	2,700	
	do	do	do	--	--	Dec.	1960	.92	64.9	--	55.2	6.69	8,000	
	do	do	do	--	--	Feb.	1962	1/1.00	64.1	--	2/55.2	2.60	3,120	
	do	do	do	--	--	Jan.	1963	.92	61.6	--	58.0	8.42	10,640	
	do	do	do	--	--	Jan.	1964	1.00	63.59	--	57.8	4/-6.19	--	
	do	do	do	--	--	Jan.	1965	1.00	59.88	--	65.44	11.49	1,048	
	do	do	do	--	--	Mar.	1969	4.17	57.58	--	5/65.44	1.71	112	
35-48b	Hale Creek "F"	Trib. of Hale Creek	do	.063	.060	Oct.	1955	--	19.4	*1.470	--	--	--	SCS
	do	do	do	--	--	Nov.	1956	1.08	18.6	*1.409	77.2	13.30	22,400	
	do	do	do	--	--	Sept.	1957	.83	17.7	*1.341	57.8	17.90	22,900	
	do	do	do	--	--	Sept.	1958	1.00	17.6	*1.333	72.8	1.50	2,400	
	do	do	do	--	--	Jan.	1960	1.33	17.1	*1.295	44.5	5.83	5,600	
	do	do	do	--	--	Dec.	1960	.92	16.3	*1.235	56.5	15.30	18,900	
	do	do	do	--	--	Feb.	1962	1.17	16.1	*1.220	2/56.5	2.67	3,300	
	do	do	do	--	--	Jan.	1963	.92	15.4	*1.167	60.3	12.17	16,000	
	do	do	do	--	--	Jan.	1964	1.00	--	--	56.0	0	0	
	do	do	do	--	--	Jan.	1965	1.00	14.7	*1.114	54.3	11.43	13,900	
	do	do	do	--	--	Jan.	1966	1.00	13.7	*1.038	54.3	17.32	20,900	
	do	do	do	--	--	Jan.	1967	1.0	14.361	1.088	54.3	4/-7.767	--	
	do	do	do	--	--	Jan.	1968	1.0	13.793	1.045	54.3	9.467	11,196	
	do	do	do	--	--	Feb.	1969	1.08	13.751	1.042	5/55.5	.65	30,756	
35-49b	Hale Creek "R"	do	do	.819	.796	Oct.	1955	--	103	*.595	--	--	--	SCS
	do	do	do	--	--	Oct.	1956	1.00	100	*.578	66.4	4.17	6,000	
	do	do	do	--	--	Sept.	1957	.92	97.4	*.563	64.4	3.98	5,600	
	do	do	do	--	--	Sept.	1958	1.00	97.2	*.562	64.8	.40	600	
	do	do	do	--	--	Jan.	1960	1.33	94.6	*.547	55.6	2.39	2,900	
	do	do	do	--	--	Dec.	1960	.92	93.2	*.539	55.7	1.93	2,300	
	do	do	do	--	--	Feb.	1962	1.17	90.8	*.525	2/55.7	2.61	3,200	
	do	do	do	--	--	Jan.	1963	.92	88.7	*.513	59.4	4.21	5,400	
	do	do	do	--	--	Jan.	1964	1.00	--	--	51.1	4/-2.02	--	
	do	do	do	--	--	Jan.	1965	1.00	86.0	*.497	49.9	5.46	5,900	
	do	do	do	--	--	Jan.	1966	1.00	80.5	*.465	49.9	6.86	7,500	
	do	do	do	--	--	Feb.	1969	1.08	73.867	.423	5/60.29	1.055	13,419	
35-50	Miles Wallace Pond	Platte	Cozad, Nebr.	.81	.81	--	1939	--	7/1.61	--	--	--	--	SCS
	do	do	do	--	--	Sept.	1964	25	.72	--	104	.05	113	
35-51	Gayle Miller Pond	do	do	.94	.94	--	1948	--	16.18	--	--	--	--	SCS
	do	do	do	--	--	Sept.	1964	16	13.06	--	70	.21	320	
35-52	Kenneth Smith Pond	do	do	.49	.49	--	1940	--	5.97	--	--	--	--	SCS
	do	do	do	--	--	Oct.	1964	24	4.99	--	95	.08	166	
35-53	Wayne Bartleff Pond	do	do	1.13	1.13	--	1940	--	15.7	--	--	--	--	SCS
	do	do	do	--	--	Oct.	1964	24	12.7	--	95	.11	228	
35-54	Oak-Middle Creek Watershed Site 67-A	North Platte	Dwight, Nebr.	.89	.89	Jan.	1958	--	175.7	--	--	--	--	SCS
	do	do	do	--	--	Dec.	1964	7	140.8	--	[illegible]	5.61	8,064	
RIVER BASIN (NIOBRARA TO ABOVE BLAIR) JAMES AND BIG SIOUX RIVER BASINS														
36-1	Split Rock	Trib. of Big Sioux River	Ihlen, Minn.	41.3	41.1	July	1938	--	899.0	*.314	--	--	--	SCS
	do	do	do	--	--	June	1949	10.9	692.1	*.242	42.8	.462	431	
36-2a	C. A. Stiles	Unnamed	Washta, Iowa	.593	.572	Dec.	1940	--	78.0	*.394	--	--	--	SCS
	do	do	do	--	--	Mar.	1949	8.3	70.7	*.347	46.9	1.54	1,580	
	do	do	do	--	--	Sept.	1950	1.5	69.1	*.335	*57.8	1.82	2,300	
	do	do	do	--	--	Feb.	1953	2.4	67.7	*.326	57.8	.98	1,240	
36-3	Farmers' Ditch Old Desilting Basin	Farmers' Ditch	Bronson, Iowa	22.9	21.4	Apr.	1941	--	8/674	*.178	--	--	--	SCS
	do	do	do	--	--	Feb.	1945	3.8	275	*.073	68.3	4.91	7,300	
36-4	Strunk Farm Pond	Trib. of Clay Creek	Mayfield, S. Dak.	.408	.407	July	1947	--	4.09	*.157	--	--	--	SCS
	do	do	do	--	--	July	1953	6	3.37	*.129	*65	.30	411	
36-5	Broderson Farm Pond	do	Volin, S. Dak.	2.46	2.45	July	1951	--	25.19	*.160	--	--	--	SCS
	do	do	do	--	--	July	1953	2	20.10	*.128	*55	1.04	1,246	
36-6	Masters Upper Reservoir	Unnamed	Mapleton, Iowa	.178	.169	Dec.	1949	--	37.95	*.949	--	--	--	SCS
	do	do	do	--	--	Sept.	1950	.79	9/38.20	*.955	--	--	--	
	do	do	do	--	--	Aug.	1951	.87	35.25	*.881	*77.7	20.1	34,015	
	do	do	do	--	--	Apr.	1952	.73	34.80	*.870	*77.7	3.67	6,211	
	do	do	do	--	--	Nov.	1952	.60	34.80	*.870	77.7	--	--	
36-7	Hepper Main	do	do	.196	.186	Mar.	1949	--	38.1	*.866	--	--	--	SCS
	do	do	do	--	--	July	1950	1.37	37.8	*.859	*76.1	1.25	2,070	
	do	do	do	--	--	July	1951	1.01	34.4	*.782	*76.1	17.96	29,768	
	do	do	do	--	--	Nov.	1952	1.31	34.4	*.782	76.1	--	--	
36-8	Hepper Southwest	do	do	.075	.069	Mar.	1949	--	8.45	*.497	--	--	--	SCS
	do	do	do	--	--	Aug.	1950	1.42	8.45	*.497	*72.7	0	0	
	do	do	do	--	--	Aug.	1951	.98	7.88	*.464	*72.7	8.41	13,319	
	do	do	do	--	--	Nov.	1952	1.30	7.88	*.464	72.7	0	0	

1/ Adjusted to correct previous years error.
2/ Based on December 1960 sample.
3/ Pond dry when surveyed.
4/ Minus (-) indicates scour or compaction. (treated as negative sediment).
5/ Weight determined by gamma probe.
6/ Includes upstream structures.
7/ Water supply pool capacity. Reservoir capacity is greater at spillway crest elevation.
8/ Conservation pool capacity. Flood pool capacity is 1,952 ac.-ft.
9/ Increase in capacity in 1950 was due to settlement of dam.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
	RIVER BASIN (NIOBRARA TO ABOVE BLAIR) JAMES AND BIG SIOUX RIVER BASINS (Continued)												
36-9	Theobold Main	Unnamed	Anthon, Iowa	.483	.442	June 1949	—	103.2	*1.053	—	—	—	SCS
	do	do	do	—	—	Aug. 1950	1.15	96.0	*.959	*72.3	18.1	28,502	
	do	do	do	—	—	May 1951	.77	93.5	*.954	*72.3	1.36	2,142	
	do	do	do	—	—	Oct. 1952	1.38	89.2	*.910	72.3	7.13	11,228	
36-10	Theobold Lateral C.	do	do	.250	.234	Nov. 1948	—	37.5	*.735	—	—	—	SCS
	do	do	do	—	—	July 1950	1.70	34.5	*.676	*67.9	7.65	11,313	
	do	do	do	—	—	May 1951	.83	34.3	*.673	*67.9	1.08	1,597	
	do	do	do	—	—	May 1952	.94	33.5	*.657	67.9	3.60	5,326	
36-11	Theobold Lateral D.	do	do	.098	.089	Nov. 1948	—	19.5	*.975	—	—	—	SCS
	do	do	do	—	—	July 1950	1.71	18.9	*.965	*73.1	3.63	5,780	
	do	do	do	—	—	May 1951	.82	18.7	*.935	*73.1	2.86	4,560	
	do	do	do	—	—	Oct. 1952	1.42	18.6	*.930	73.1	.72	1,146	
36-12a	Mortenson	do	Cherokee, Iowa	1.39	1.37	Oct. 1950	—	229.0	*.718	—	—	—	SCS
36-13	Bartlett	Green Creek	Alcester, S. Dak.	.234	.234	Nov. 1936	—	12.84	1.035	—	—	—	SCS
	do	do	do	—	—	Nov. 1955	19	6.86	.553	77.0	1.35	2,264	
36-14b	Lake Mitchell	Firesteel Creek	Mitchell, S. Dak.	584	496	Oct. 1928	—	10,943	.49	—	—	—	SCS
	do	do	do	—	—	July 1948	19.8	10,380	.47	*46.79	.057	58	
	do	do	do	—	—	Aug. 1958	10.0	9,798	.44	*46.79	.117	119	
	do	do	do	—	—	July 1969	11.0	9,235	.42	65.47	.103	231	
36-15	Salem Dam	Silver Creek	Sioux Falls, S. Dak.	1.158	1.158	Aug. 1950	—	12.79	.206	—	—	—	SCS
	do	do	do	—	—	Aug. 1955	5.0	8.16	.132	*60	.80	1,045.44	
36-16	Scott No. 2	Scott Creek	Alcester, S. Dak.	.152	.152	Oct. 1936	—	7.36	.909	—	—	—	SCS
	do	do	do	—	—	Apr. 1955	18.5	3.14	.380	36.4	1.52	1,067.2	
36-17	Lake Vermillion	Vermillion	Montrose, S. Dak.	402.9	402.0	Mar. 1939	—	5,173.7	.297	—	—	—	SCS
	do	do	do	—	—	June 1964	5.25	4,927.5	.283	35	.117	89	
36-18	Vermillion Watershed No. A	Trib. of Vermillion	Salem, S. Dak.	.075	.075	Aug. 1955	—	3.20	1.000	—	—	—	SCS
	do	do	do	—	—	Nov. 1965	10.3	2.67	.836	*50	.68	760	
36-19	Vermillion Watershed No. B	do	do	.237	.237	Oct. 1950	—	6.48	.639	—	—	—	SCS
	do	do	do	—	—	Nov. 1965	15	5.06	.491	*50	.60	435	
36-20	Lake Okabena	Trib. of Okabena Creek	Worthington, Minn.	17.40	16.27	— *1897	—	5,183	2.234	—	—	—	SCS
	do	do	do	—	—	Mar. 1963	65	1/5,100	2.198	26	.07	492	
36-21a	Scott Creek #11	Scott Creek	Alcester, S. Dak.	1.85	1.65	Oct. 1956	—	196.64	2.55	—	—	—	SCS
	do	do	do	—	—	Apr. 1965	10.5	192.7	2.50	*65	.26	368	
	do	do	do	—	—	May 1970	5.1	179.75	2.33	—	1.785	2,527	
36-22	Richmond Lake	Foot Creek	Aberdeen, S. Dak.	82	73.5	Jan. 1937	—	9,880.9	7.53	—	—	—	SCS
	do	do	do	—	—	Sept. 1969	32.75	8,971.08	6.84	51.8	.378	426	
	MISSOURI RIVER BASIN (ABOVE PIERRE TO NIOBRARA) Niobrara and White River Basins												
37-1a	Elkins Stock Pond No. 1	Br. of Prosemann Creek	Hayes, S. Dak.	.58	.57	May 1907	—	18.61	—	—	—	—	SCS
	do	do	do	—	—	June 1937	30	15.78	—	41.3	.16	144	
37-2	Elkins Stock Pond No. 2	do	do	.33	.33	May 1911	—	4.36	—	—	—	—	SCS
	do	do	do	—	—	June 1937	26.0	3.18	—	36.6	.136	108	
37-3	Land Utilization Project No. 226-1.	Trib. of Bad River	Pierre, S. Dak.	.203	.197	Mar. 1936	—	16.7	—	—	—	—	SCS
	do	do	do	—	—	July 1945	9.3	15.2	—	—	.822	—	
37-4	Land Utilization Project No. 226-2.	Trib. of Missouri River	do	.995	.981	Mar. 1936	—	34.0	—	—	—	—	SCS
	do	do	do	—	—	July 1945	9.3	29.[illegible]	—	—	.458	—	
37-5	Land Utilization Project No. 226-4.	Trib. of Bad River	do	.768	.762	May 1936	—	13.[illegible]	—	—	—	—	SCS
	do	do	do	—	—	July 1945	9.2	9.9	—	—	.539	—	
37-6	Land Utilization Project No. 226-6.	Trib. of Missouri River	do	2.555	2.541	Apr. 1936	—	31.8	—	—	—	—	SCS
	do	do	do	—	—	July 1945	9.3	25.1	—	—	.285	—	
37-7	Land Utilization Project No. 226-13.	Trib. of Bad River	do	.166	.163	Oct. 1936	—	9.1	—	—	—	—	SCS
	do	do	do	—	—	July 1945	8.7	8.3	—	—	.521	—	
37-8	Land Utilization Project No. 226-21.	Trib. of Missouri River	do	.245	.234	Oct. 1936	—	28.9	—	—	—	—	SCS
	do	do	do	—	—	July 1945	8.7	26.1	—	—	1.309	—	
37-9	Land Utilization Project No. 226-22.	Trib. of Bad River	do	.514	.511	Nov. 1936	—	9.2	—	—	—	—	SCS
	do	do	do	—	—	July 1945	8.7	8.3	—	—	.200	—	

37-10	Land Utilization Project No. 226-25.	Trib. of Missouri River	do	.147	.144	Nov.	1937	—	9.3	—	—	—	—	SCS
	do	do	do	—	—	July	1945	7.7	8.4	—	—	.799	—	
37-11	Land Utilization Project No. 226-31.	do	do	.225	.220	Mar.	1937	—	13.9	—	—	—	—	SCS
	do	do	do	—	—	July	1945	8.3	12.3	—	—	.859	—	
37-12	Land Utilization Project No. 226-32.	do	do	.477	.472	Mar.	1937	—	14.1	—	—	—	—	SCS
	do	do	do	—	—	July	1945	8.3	12.1	—	—	.508	—	
37-13	Land Utilization Project No. 226-34.	do	do	1.222	1.209	June	1937	—	44.3	—	—	—	—	SCS
	do	do	do	—	—	July	1945	8.1	40.0	—	—	.443	—	
37-14	Land Utilization Project No. 226-35.	do	do	.508	.502	May	1938	—	18.4	—	—	—	—	SCS
	do	do	do	—	—	July	1945	7.2	16.1	—	—	.631	—	
37-15	Land Utilization Project No. 243-1.	do	do	.108	.106	Nov.	1937	—	2.7	—	—	—	—	SCS
	do	do	do	—	—	July	1945	7.7	2.0	—	—	.011	—	
37-16	Land Utilization Project No. 243-2.	Unnamed Stream (interior drainage)	do	.134	.131	Nov.	1937	—	5.2	—	—	—	—	SCS
	do	do	do	—	—	July	1945	7.7	3.0	—	—	2.206	—	
37-17	Land Utilization Project No. 243-5.	Trib. of Missouri River	do	.072	.070	Nov.	1937	—	2.6	—	—	—	—	SCS
	do	do	do	—	—	July	1945	7.7	2.2	—	—	.700	—	
37-18	Land Utilization Project No. 243-6.	Trib. of Bad River	do	.117	.116	Nov.	1937	—	3.2	—	—	—	—	SCS
	do	do	do	—	—	July	1945	7.7	2.8	—	—	.474	—	
37-19	Land Utilization Project No. 243-10.	Trib. of Missouri River	do	.533	.532	June	1938	—	3.7	—	—	—	—	SCS
	do	do	do	—	—	July	1945	7.1	2.9	—	—	.222	—	
37-20	Land Utilization Project No. 243-11.	do	do	.339	.338	July	1938	—	4.5	—	—	—	—	SCS
	do	do	do	—	—	July	1945	7.0	3.9	—	—	.266	—	
37-21	Harris Reservoir North	Unnamed	Gordon, Nebr.	.577	.573	May	1947	—	7.3	—	—	—	—	SCS
	do	do	do	—	—	Nov.	1956	9.5	5.57	—	*70	.315	480	
37-22	Harris Reservoir South	do	do	11.9	11.85	May	1947	—	18.64	—	—	—	—	SCS
	do	do	do	—	—	Nov.	1956	9.5	15.77	—	*70	.025	38	
37-23	Johnson Reservoir	do	do	.856	.849	—	1946	—	16.25	—	—	—	—	SCS
	do	do	do	—	—	Nov.	1956	10.3	12.39	—	*70	.435	663	
37-24a	Lake Dante	Trib. of Choteau Creek	Wagner, S. Dak.	2.89	2.89	Nov.	1937	—	260.73	.352	—	—	—	SCS
	do	do	do	—	—	Oct.	1955	17.9	230.95	.312	*60	.578	756	
37-25	Kegara	Trib. of Missouri River	do	10.3	10.3	July	1937	—	231.0	.840	—	—	—	SCS
	do	do	do	—	—	July	1965	28	200.32	.728	50	.107	115	

MISSOURI RIVER BASIN (MOBRIDGE TO ABOVE PIERRE)

Cheyenne and Belle Fourche River Basins

38-1	Johnson's Stock Pond	Trib. of L. Cheyenne River	Gettysburg, S. Dak.	.191	.188	—	1912	—	2.56	—	—	—	—	SCS
	do	do	do	—	—	June	1937	25.0	1.85	—	58.1	.16	202	
38-2	Bartel Stock Pond	Little Cheyenne Creek	do	.609	.606	—	1908	—	4.30	—	—	—	—	SCS
	do	do	do	—	—	June	1937	29.0	2.80	—	53.6	.08	93.4	
38-3a	W-14 (Anderson)	Trib. of Fourmile Creek	Vale, S. Dak.	.054	.054	Nov.	1956	—	5.60	1.42	—	—	—	ARS
	do	do	do	—	—	Oct.	1958	1.9	5.14	1.30	70	4.48	6,708	
	do	do	do	—	—	Dec.	1962	4.2	4.51	1.14	70	2.78	4,158	
	do	do	do	—	—	Apr.	1967	4.3	4.36	1.11	70	.65	967	
38-4a	W-9 (Swenk)	Sulphur Creek	Sulphur, S. Dak.	1.27	1.27	Nov.	1951	—	390.9	10.35	—	—	—	ARS
	do	do	do	—	—	June	1961	9.6	334	9.85	84	1.38	2,525	
	do	do	do	—	—	June	1969	8.0	315.3	9.3	84	1.84	3,367	
38-5	Angostura Reservoir	Cheyenne River	Hot Springs, S. Dak.	9,100	9,093	Oct.	1949	—	159,919	.866	—	—	—	BR
	do	do	do	—	—	Sept.	1965	16	138,761	.752	56.4	.145	178	
38-6	Sturgis Watershed No. 1	Alkali Creek	Sturgis, S. Dak.	.339	.339	Oct.	1965	—	—	—	—	—	—	FS
	do	do	do	—	—	Oct.	1966	1.0	—	—	92.2	2/—	.019	
	do	do	do	—	—	Oct.	1967	1.0	—	—	92.2	2/—	.687	
	do	do	do	—	—	Oct.	1968	1.0	—	—	92.2	2/—	.044	
	do	do	do	—	—	Oct.	1969	1.0	—	—	92.2	2/—	.252	
	do	do	do	—	—	Oct.	1970	1.0	—	—	92.2	2/—	.495	
38-7	Sturgis Watershed No. 2	do	do	.139	.139	Oct.	1965	—	—	—	—	—	—	FS
	do	do	do	—	—	Oct.	1966	1.0	—	—	92.2	2/—	.05	
	do	do	do	—	—	Oct.	1967	1.0	—	—	92.2	.001	2.218	
	do	do	do	—	—	Oct.	1968	1.0	—	—	92.2	2/—	.04	
	do	do	do	—	—	Oct.	1969	1.0	—	—	92.2	2/—	.036	
	do	do	do	—	—	Oct.	1970	1.0	—	—	92.2	.004	8.705	
38-8	Sturgis Watershed No. 3	do	do	.297	.297	Oct.	1965	—	—	—	—	—	—	FS
	do	do	do	—	—	Oct.	1966	1.0	—	—	92.2	2/—	.106	
	do	do	do	—	—	Oct.	1967	1.0	—	—	92.2	2/—	.404	
	do	do	do	—	—	Oct.	1968	1.0	—	—	92.2	2/—	.074	
	do	do	do	—	—	Oct.	1969	1.0	—	—	92.2	2/—	.022	
	do	do	do	—	—	Oct.	1970	1.0	—	—	92.2	.001	2.438	
38-9	Canyon Lake	Rapid Creek	Rapid City, S. Dak.	385	3/66.35	Sept.	1933	—	300.77	.011	—	—	—	SCS
	do	do	do	—	—	Oct.	1970	37	221.75	.008	65	4/.032	4/50	

1/ 837 ac.-ft. of storage created by dredging.
2/ Less than .001. Material removed from Weir Pond facility periodically.
3/ Pactola Reservoir controls about 320 sq. mi.
4/ 9.9 ac.-ft. sediment, removed in 1956, not included.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
MISSOURI RIVER BASIN (MOBRIDGE TO ABOVE PIERRE) Cheyenne and Belle Fourche River Basins (Continued)													
38-10	Barrel Detention Dam	Unnamed	New Castle, Wy.	6.15	4.83	Aug. 1968	—	454.6	*6.9	—	—	—	SCS
	do	do	do	—	—	Dec. 1970	2.25	438.4	*6.7	86	1.49	2,799	
38-11	New Underwood Reservoir	do	New Underwood, S. Dak.	2.97	2.94	— 1935	—	138.5	—	—	—	—	SCS
	do	do	do	—	—	Aug. 1970	35	120.2	—	*60	.177	231	
MISSOURI RIVER BASIN (WILLISTON TO MOBRIDGE) Moreau, Grand, Cannonball, Heart, and Little Missouri River Basins													
39-1	Frederick Stock Dam	Louse Creek	Flasher, N. Dak.	1.17	1.17	-- 1935	—	1.06	*.021	—	—	—	SCS
	do	do	do	—	—	Aug. 1956	21	.864	*.017	*35	.00797	6.07	
39-2a	Hiddenwood Lake	Hiddenwood Creek	Selby, S. Dak.	31.8	27.2	May 1926	—	1/207.78	.549	—	—	—	SCS
	do	do	do	—	--	Sept. 1959	33.3	165.70	.438	55	.047	56	
39-3	Kashiesh Reservoir	Trib. of Moreau	Sorum, S. Dak.	2.20	1.97	Aug. 1924	—	389.80	—	—	—	—	SCS
	do	do	do	—	—	Aug. 1964	40	380.75	--	*60	.12	156.8	
39-4	Clark Reservoir	do	do	3.56	2.56	Aug. 1949	—	62.00	—	—	--	—	SCS
	do	do	do	—	—	Aug. 1964	15	56.15	—	*60	.15	196	
39-5	Wenner Reservoir	Trib. of Grand	Lemmon, S. Dak.	.493	.493	Aug. 1951	--	35.96	3.182	—	—	—	SCS
	do	do	do	—	—	Aug. 1964	13	33.96	3.005	*60	.30	392	
39-6	Cole Reservoir	do	Bison, S. Dak.	4.66	2.21	Aug. 1937	—	161.1	—	—	—	—	SCS
	do	do	do	—	—	Aug. 1964	27	144.01	—	*60	.29	379	
39-7	Van Oosting Dam	Trib. of Missouri River	Hensler, N. Dak.	1.78	1.76	Apr. 1950	—	171.5	*3.618	—	--	--	SCS
	do	do	do	--	—	Oct. 1964	14.5	167.0	*3.523	60.4	2/.176	2/231.5	
39-8	Battle Creek Detention Dam	Battle Creek	Hulett, Wy.	18.44	18.44	Nov. 1967	--	620.1	*.63	—	—	—	SCS
	do	do	do	—	—	Sept. 1970	2.83	614.7	*.62	71.4	.104	161.8	
MISSOURI RIVER BASIN (ZORTMAN TO WILLISTON) Milk and Musselshell River Basins													
40-1	Yellow Water	Yellow Water Creek	Winnett, Mont.	55	54	June 1938	—	4,796	—	—	--	—	SCS
	do	do	do	—	—	Sept. 1948	10.3	4,616	—	*32	.32	224	
40-2	Fort Peck	Missouri Riv.	Fort Peck, Mont.	57,725	3/--	Oct. 1937	--	19,557,492	—	—	—	—	CE
	do	do	do	—	—	June 1950	12.67	19,233,686	—	56.7	.65	803	
	do	do	do	—	—	Sept. 1952	2.25	19,186,698	—	57.4	.53	724	
	do	do	do	—	—	June 1954	1.75	19,164,369	—	57.9	.33	471	
	do	do	do	—	--	May 1958	3.92	19,162,468	—	58.8	.013	75	
	do	do	do	—	—	June 1961	3.08	19,138,511	--	59.4	.22	338	
MISSOURI RIVER BASIN (ABOVE ZORTMAN)													
41-1	Anderson Reservoir	Trib. of Pondera Creek	Conrad, Mont.	.156	.156	— 1916	—	9.27	—	—	—	—	SCS
	do	do	do	—	--	Oct. 1955	39	8.44	—	*60	.136	183	
41-2a	Kropp	do	do	.625	.625	— 1900	—	4.70	—	—	—	—	SCS
	do	do	do	--	—	Oct. 1955	55	3.84	--	*60	.025	33	
41-3	Walston	do	do	2.96	2.96	— 1915	—	66.13	--	—	—	—	SCS
	do	do	do	—	--	Oct. 1955	40	51.71	—	*60	.122	166	
LOWER YELLOWSTONE RIVER BASIN Tongue and Powder River Basins													
42-1a	Baker Lake	Sandstone Creek	Baker, Mont.	5.20	5.01	May 1908	—	756	3.40	—	—	—	SCS
	do	do	do	—	—	May 1937	29.1	502	2.26	39	1.74	1,478	
	do	do	do	—	--	May 1955	18.0	393	1.77	45	1.21	1,553	
	do	do	do	--	--	July 1970	15.2	235	1.06	40	2.08	1,288	
42-2	Tongue River	Tongue River	Decker, Mont.	1,740	1,734.5	May 1939	—	72,510	.248	—	—	—	BR
	do	do	do	--	—	Oct. 1948	9.42	69,439	.237	70.5	.188	288.7	
42-3	Narong Reservoir	Dry Creek	Miles City, Mont.	6.92	6.92	— 1941	—	300	4.05	—	—	—	SCS
	do	do	do	—	—	— 1969	28	222	3.0	70	.403	614	

UPPER YELLOWSTONE RIVER BASIN

43-1b	Buffalo Bill	Shoshone River	Cody, Wy.	1,470	1,470	May	1910	—	455,838	.492	—	—	—	BR
	do	do	do	—	—	—	1941	31.7	439,851	.476	—	.34	—	
	do	do	do	—	—	—	1958	17.0	421,333	.455	—	.62	—	
43-2	Big Gin	Trib. of Fifteenmile Creek	Worland, Wy.	.942	.918	Oct.	1954	—	99.2	3.906	—	—	—	GS
	do	do	do	—	—	Oct.	1955	1	98.1	3.862	—	1.2	—	
	do	do	do	—	—	Oct.	1958	3	96.0	3.780	—	.8	—	
	do	do	do	—	—	Oct.	1961	3	93.2	3.669	—	1.0	—	
43-3	Fraser Draw	Fraser Draw	Moneta, Wy.	27.4	27.4	Oct.	1954	—	90.2	1.797	—	—	—	GS
	do	do	do	—	—	Oct.	1958	4	87.7	1.747	—	.02	—	
	do	do	do	—	—	Nov.	1960	2	83.3	1.699	—	.08	—	
43-4	East Fork	Graham Draw	do	.81	.81	June	1949	—	12.30	4.241	—	—	—	GS
	do	do	do	—	—	Nov.	1949	.42	11.73	4.065	—	1.68	—	
	do	do	do	—	—	May	1952	2.54	11.01	3.797	—	.35	—	
	do	do	do	—	—	Oct.	1955	3.42	10.83	3.734	—	.06	—	
	do	do	do	—	—	Oct.	1957	2.00	10.61	3.659	—	.14	—	
	do	do	do	—	—	Oct.	1960	3.00	10.58	3.648	—	.01	—	
43-5	West Fork	do	do	.38	.38	July	1948	—	3.13	3.827	—	—	—	GS
	do	do	do	—	—	Nov.	1949	1.25	2.96	3.585	—	.39	—	
	do	do	do	—	—	May	1952	2.54	2.56	3.098	—	.42	—	
	do	do	do	—	—	Oct.	1955	3.42	2.62	3.195	—	0	—	
	do	do	do	—	—	Oct.	1957	2.00	2.59	3.159	—	.05	—	
	do	do	do	—	—	Oct.	1960	3.00	2.47	3.012	—	.11	—	
43-6	Red Spires	Rock Waterhole Creek	Worland, Wy.	5.24	5.20	Oct.	1954	—	193.4	1.098	—	—	—	GS
	do	do	do	—	—	Oct.	1955	1	191.3	1.086	—	.40	—	
	do	do	do	—	—	Oct.	1958	3	176.1	1.000	—	.98	—	
	do	do	do	—	—	Oct.	1961	3	154.5	.877	—	1.38	—	
43-7	Boysen	Wind River	Thermopolis, Wy.	7,700	7,670	Oct.	1951	—	819,132	.625	—	—	—	BR
	do	do	do	—	—	Aug.	1964	12.8	802,004	.612	83	.17	307	

ARKANSAS RIVER BASIN (VAN BUREN TO LITTLE ROCK) WHITE RIVER BASIN

44-1a	Lake Bennett	East Fork Creek	Conway, Ark.	4.16	4.11	June	1935	—	496	—	—	—	—	SCS
	do	do	do	—	—	Nov.	1935	0.4	490	—	—	2.43	—	
	do	do	do	—	—	Nov.	1946	11.0	481	.130	—	.20	—	
44-2	Lake Booneville	Trib. of Petit Jean Creek	Booneville, Ark.	2.60	2.57	Mar.	1929	—	289	.127	—	—	—	SCS
	do	do	do	—	—	Nov.	1935	6.75	282	—	—	.455	—	
44-3	Lake Fort Smith	Jack and Jones Creeks	Mountainburg, Ark.	65	64	Feb.	1936	—	*13,810	—	—	—	—	SCS
	do	do	do	—	—	Apr.	1960	4.2	13,727	—	—	.31	—	
44-4	Lake Bailey	Cedar Creek	Morrilton, Ark.	15.2	15	Sept.	1937	—	629	—	—	—	—	SCS
	do	do	do	—	—	May	1960	2.7	602	—	—	.66	—	
44-5	Lake Taneycomo	White River	Branson, Mo.	4,610	4,606	Mar.	1913	—	43,980	—	—	—	—	SCS
	do	do	do	—	—	Aug.	1935	22.4	23,714	—	—	.196	—	
44-6	Nimrod	Fourche La Fave River	Nimrod, Ark.	680	652	May	1942	—	336,000	.532	—	—	—	CE
	do	do	do	—	—	Apr.	1950	7.9	4/336,000	.532	—	4/—	—	
44-7	Norfolk	North Fork River	Norfork, Ark.	1,806	1,772	June	1943	—	1,560,900	1.138	—	—	—	CE
	do	do	do	—	—	May	1950	6.9	5/1,560,900	1.138	—	5/—	—	
44-8	Cove Lake	Arkansas River Trib.	Paris, Ark.	10.90	10.66	May	1938	—	2,206	*.298	—	—	—	SCS
	do	do	do	—	—	Dec.	1953	15.5	2,141	*.290	—	.39	—	
44-9	Charleston Lake	do	Charleston, Ark.	1.03	.93	Apr.	1937	—	325	*.455	—	—	—	SCS
	do	do	do	—	—	Dec.	1953	16.66	264	*.370	—	3.94	—	
44-10	Six Mile Creek No. 2	Shaver Creek	Caulksville, Ark.	5.38	5.26	Oct.	1954	—	6/1,613.93	.469	—	—	—	SCS
	do	do	do	—	—	July	1961	6.67	6/1,608.15	.467	—	6/.17	—	
	do	do	do	—	—	Oct.	1954	—	7/309.52	.090	—	—	—	
	do	do	do	—	—	July	1961	6.67	7/301.01	.087	—	7/.24	—	
44-11	Six Mile Creek No. 5	Little Cansy Creek	do	1.90	1.81	Dec.	1954	—	6/860.98	.708	—	—	—	SCS
	do	do	do	—	—	July	1961	6.63	6/854.63	.703	—	6/.53	—	
	do	do	do	—	—	Dec.	1954	—	7/221.96	.183	—	—	—	
	do	do	do	—	—	July	1961	6.63	7/215.77	.177	—	7/.51	—	
44-12	Six Mile Creek No. 23	Trib. of Hurricane Creek	Peter Pender, Ark.	4.16	3.91	July	1955	—	6/989.25	.372	—	—	—	SCS
	do	do	do	—	—	July	1961	6.00	6/983.27	.369	—	6/.26	—	
	do	do	do	—	—	July	1955	—	7/75.92	.029	—	—	—	
	do	do	do	—	—	July	1961	6.00	7/70.64	.027	—	7/.23	—	

1/ Conservation pool capacity. Reservoir has greater capacity at spillway crest elevation.
2/ Includes 0.03 ac.-ft. above crest deposits.
3/ 1937-1955=39,097 sq. mi.; 1956-1961=34,692 sq. mi.
4/ Suspended - load inflow was 608 ac.-ft.; suspended - load outflow was 200 ac.-ft. during period. Deposits too small to measure by range survey.
5/ Sediment inflow volume was computed to be 2,350 ac.-ft.; much of this probably settled out over a large area in deposits too thin to be measured accurately by echo sounders.
6/ Both sediment and flood pools.
7/ Sediment pool only.
* Estimated or assumed.

SUMMARY OF

RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

ARKANSAS RIVER BASIN (VAN BUREN TO LITTLE ROCK) WHITE RIVER BASIN (Continued)

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
44-13	Dardanelle Reservoir	Arkansas River	Dardanelle, Ark.	1/153,703	11,333	Oct. 1964	—	460,300	2/.0532	—	—	—	CE
	do	do	do	—	—	Oct. 1965	1.08	452,600	2/.0532	*80	0.626	1,090	
	do	do	do	—	—	Apr. 1968	2.50	443,500	2/.0540	80	.321	560	
44-14	Six Mile Creek Site No. 6	Six Mile Creek	Chismville, Ark.	4.14	4.07	Oct. 1954	—	1,304.03	.446	—	—	—	SCS
	do	do	do	—	—	May 1964	9.58	1,275.62	.436	68.8	.73	1,093	
	do	do	do	—	—	Oct. 1970	6.42	1,253.26	.429	76.1	.86	1,425	

ARKANSAS RIVER BASIN (TULSA TO VAN BUREN)
Grand, Verdigris, and Lower Canadian River Basins

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA TOTAL	DRAINAGE AREA NET	DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO	SPECIFIC WEIGHT	AC.-FT.	TONS	AGENCY SUPPLYING DATA
45-1	Lake Wedington	Trib. of Illinois River	Fayetteville, Ark.	4.06	3.92	July 1937	—	1,260	—	—	—	—	SCS
	do	do	do	—	—	Aug. 1947	10.1	1,249	—	—	.278	—	
45-2	Wilson	Wilson Creek	do	2.35	2.30	Oct. 1930	—	522	—	—	—	—	SCS
	do	do	do	—	—	June 1940	9.75	517	—	—	.23	—	
45-3	Lake Sapulpa	Euchee Creek	Sapulpa, Okla.	8.72	8.57	— 1913	—	1,094	—	—	—	—	SCS
	do	do	do	—	—	Dec. 1935	22.5	911	—	—	.949	—	
	do	do	do	—	—	Dec. 1946	11.0	842	—	—	.732	—	
45-4b	Brown Lake	Peaceable Creek	McAlester, Okla.	20.9	19.9	May 1943	—	4,995	.640	—	—	—	SCS
	do	do	do	—	—	July 1952	9.2	4,660	.597	54.2	1.83	2,160	
	do	do	do	—	—	Apr. 1963	10.75	4,525	.580	—	.63	743	
45-5	Lake McAlester	Bull Creek	do	30.7	28.2	— 1919	—	18,397	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1941	22	17,509	—	—	1.43	—	
45-6	Lake Okmulgee	Salt Creek	Okmulgee, Okla.	40.1	39.2	— 1928	—	10,355	—	—	—	—	SCS
	do	do	do	—	—	Apr. 1950	22	9,896	—	—	.532	—	
45-7a	Shawnee Lake	Deer Creek	Shawnee, Okla.	21.2	18.9	Aug. 1937 3/	—	3/22,600	—	—	—	—	SCS
								24,468	—	—	—	—	
	do	do	do	—	—	Dec. 1947 4/	12.3	5/23,816	—	94.72	2.8	5,776	
								23,429	—	—	—	—	
	do	do	do	—	—	Mar. 1957	9.2	5/22,327	—	94.72	2.7	5,339	
45-8	Taft Lake	Br. of Pecan Creek	Taft, Okla.	2.3	2.2	— 1937	—	580	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1947	10	574	—	—	.29	—	
45-9	Hiwassee Lake	Trib. of Deep Fk. Creek	Arcadia, Okla.	4.58	4.29	— 1936	—	2,341	—	—	—	—	SCS
	do	do	do	—	—	Apr. 1950	14	2,158	—	—	3.05	—	
45-10	Lake Carleton	Fourche Maline Creek	Wilburton, Okla.	19.8	19.7	— 1933	—	6/570	—	—	—	—	SCS
	do	do	do	—	—	July 1947	14	511	—	—	.21	—	
45-11	Holdenville City Lake	Beemore Creek	Holdenville, Okla.	8.95	8.30	June 1931	—	9,844	—	—	—	—	SCS
	do	do	do	—	—	Apr. 1950	18.8	9,030	—	—	5.22	—	
45-12	Pretty Water Lake	Pretty Water Creek	Sapulpa, Okla.	2.43	2.40	Mar. 1936	—	344	—	—	—	—	SCS
	do	do	do	—	—	Dec. 1946	10.75	322	—	—	.85	—	
45-13	Greenleaf Lake	Big Greenleaf Creek	Muskogee, Okla.	81.25	79.84	Mar. 1937	—	13,005	—	—	—	—	SCS
	do	do	do	—	—	Nov. 1941	4.7	12,828	—	—	.47	—	
45-15	Kirk Lake	Unnamed	Iola, Kans.	2.41	2.36	— 1897	—	111	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1939	42	69	—	48.10	.43	450	
45-16	Lowell	Sprg. R. & Shoal Creek	Baxter Springs, Kans.	2,210.0	2,208.6	Mar. 1905	—	10,404	—	—	—	—	SCS
	do	do	do	—	—	Aug. 1939	34.4	7,580	—	*60	.04	52.3	
45-17	Neosho County State Lake (Lake McKinley).	Small Trib. of Neosho River	Parsons, Kans.	3.38	3.24	July 1927	—	680	—	—	—	—	SCS
	do	do	do	—	—	Aug. 1939	12.1	651	—	*60	.74	967	
45-18	Lake Claremore	Dog Creek	Claremore, Okla.	56.44	55.70	May 1931	—	4,258	—	—	—	—	SCS
	do	do	do	—	—	Oct. 1939	8.4	3,909	—	51.17	.749	835	
45-19	Lake Spavinaw	Spavinaw Creek	Spavinaw, Okla.	400.0	397.2	Apr. 1924	—	31,686	—	—	—	—	SCS
	do	do	do	—	—	July 1935	11	30,509	—	—	.269	—	
45-20	Kennamer Lake	Trib. of Pryors Creek	Chelsea, Okla.	.28	.27	— 1923	—	28	—	—	—	—	SCS
	do	do	do	—	—	— 1939	16	26.5	—	—	.35	—	
45-21	State Fish Hatchery Lake	Happy Creek	Claremore, Okla.	1.21	1.20	— 1934	—	34.0	—	—	—	—	SCS
	do	do	do	—	—	— 1939	5	32.9	—	—	.18	—	
45-22	Lake Scarbow	Trib. of Pryor Creek	Pryor, Okla.	3.07	3.04	— 1931	—	123	—	—	—	—	SCS
	do	do	do	—	—	— 1939	8	111.9	—	—	.46	—	
45-23	Wewoka Lake	Coon Cr. or Spring Cr.	Wewoka, Okla.	16.27	15.72	July 1925	—	7/4,961	1.028	—	—	—	SCS
	do	do	do	—	—	Apr. 1945	19.6	4,489	.930	—	1.51	—	
	do	do	do	—	—	Feb. 1954	8.0	4,357	.903	—	8/1.05	—	
45-24	Wetumka City Lake	Salt Creek	Wetumka, Okla.	4.15	3.88	Jan. 1931	—	2,076	1.341	—	—	—	SCS
	do	do	do	—	—	Feb. 1954	23.2	1,898	1.226	—	1.98	—	
45-25	Adair	N. Caney River	Sedan, Kans.	.33	.33	Mar. 1953	—	20.9	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1960	7.5	20.2	—	55	.28	335	
45-26	Bevoy	Big Caney River	Cedar Vale, Kans.	.16	.16	Jan. 1938	—	22.8	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1960	22.7	21.2	—	60	.44	576	

45-27	Buffalo	Bee Creek	Havana, Kans.	.26	.26	— 1934	—	51.7	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1960	26	48.5	—	50	.45	490	
45-28a	Foster	N. Caney River	Sedan, Kans.	2.25	2.25	Jan. 1951	—	484	—	—	—	—	SCS
	do	do	do	—	—	Aug. 1960	9.5	477.8	—	55	.29	347	
45-29	Rouse	Caney River	Hewins, Kans.	.25	.25	Sept. 1936	—	10.66	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1960	24	9.26	—	60	.23	300	
45-30	Herbert Miles Pond	Unnamed	Lebo, Kans.	.194	.192	Jan. 1939	—	10.59	—	—	—	—	SCS
	do	do	do	—	—	July 1957	18.5	8.56	—	*60	9/.60	9/810.2	
45-31	Kahola	Kahola Creek	Dunlap, Kans.	15.9	15.3	Dec. 1936	—	6,492	*1.591	—	—	—	ARS
	do	do	do	—	—	Apr. 1954	18.3	5,935	*1.455	51.6	1.99	2,236	
45-32	Mills	Big Caney River	Sedan, Kans.	.2	.2	— 1920	—	23.9	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1960	40	21.6	—	70	.28	427	
45-33	Payer Pond	Unnamed	Burlington, Kans.	.345	.345	Jan. 1938	—	54.97	—	—	—	—	SCS
	do	do	do	—	—	July 1957	19.5	44.12	—	57	1.61	1,998.8	
45-34	Staats	Middle Caney River	Cedarvale, Kans.	.49	.49	Jan. 1943	—	81.46	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1960	17.7	77.94	—	60	.41	536	
45-35	Hulah	Caney River	Hulah, Okla.	732	712	Feb. 1950	—	295,130	1.109	—	—	—	CE
	do	do	do	—	—	June 1958	8.4	292,565	1.099	44.2	.429	413	
45-36a	Double Creek Site No. 5	do	Ramona, Okla.	2.39	2.36	Feb. 1955	—	747.36	.83	—	—	—	SCS
	do	do	do	—	—	Apr. 1964	9.2	734.29	.81	54.3	.60	710	
	do	do	do	—	—	Sept. 1969	5.5	730.53	.81	54.3	.29	343	
45-37	Waldron Lake	Square Rock Creek	Waldron, Ark.	7.65	7.58	Mar. 1943	—	361.44	.059	—	—	—	SCS
	do	do	do	—	—	Oct. 1961	18.58	325.78	.053	—	.25	—	
45-38	Howard City Lake	Game Creek	Howard, Kans.	10.16	10.06	Sept. 1936	—	771.0	.316	—	—	—	SCS
	do	do	do	—	—	Sept. 1965	28.9	604.7	.248	*60	.57	745	
45-39	Heyburn Reservoir	Polecat Creek	Heyburn, Okla.	123	117	Mar. 1950	—	59,690	1.447	—	—	—	CE
	do	do	do	—	—	Dec. 1959	9.8	57,270	1.389	77.23	2.08	3,499	
45-40	Toronto Lake	Verdigris	Toronto, Kans.	730	714	Mar. 1960	—	195,300	.559	—	—	—	CE
	do	do	do	—	—	May 1966	6.21	192,060	.550	90.4	.731	802	
45-41	Eufaula Lake	Canadian	Whitefield, Okla.	47,522	13,693	Feb. 1964	—	3,848,000	.89	—	—	—	CE
	do	do	do	—	—	June 1969	5.33	3,798,400	.88	62.2	.68	921	
45-42	Hatch	Trib. Neosho River	Burlington, Kans.	.19	.18	Oct. 1939	—	4.74	.07	—	—	—	SCS
	do	do	do	—	—	Aug. 1967	28	3.54	.05	*60	.24	314	
45-43	Mound Valley Experimental Sta.	Trib. of Pumpkin Creek	Mound Valley, Kans.	.42	.42	June 1954	—	6.90	.032	—	—	—	SCS
	do	do	do	—	—	June 1968	14	4.77	.022	*60	.36	470	
45-44	Craner	Trib. Verdigris River	Edna, Kans.	.22	.22	Nov. 1953	—	5.01	.044	—	—	—	SCS
	do	do	do	—	—	June 1968	14.6	3.92	.034	*60	.32	418	
45-45	Scott	Trib. Neosho	Parsons, Kans.	.23	.23	Dec. 1956	—	12.0	.10	—	—	—	SCS
	do	do	do	—	—	June 1968	11.8	11.1	.093	*60	.35	457	
45-46	Fletcher	Big Creek	Gridley, Kans.	.20	.19	— 1934	—	24.41	.33	—	—	—	SCS
	do	do	do	—	—	Aug. 1967	33	20.1	.27	*60	.68	888	
45-47	Iosh Pond	do	do	.38	.37	— 1927	—	15.37	.110	—	—	—	SCS
	do	do	do	—	—	Aug. 1967	40	8.72	.062	*60	.46	601	
45-48	Ness Pond	Trib. Duck Creek	Elk City, Kans.	1.75	1.73	— 1933	—	53.04	.080	—	—	—	SCS
	do	do	do	—	—	July 1967	34	29.69	.045	*60	.40	522	
45-49	Olenhouse	do	La Fontaine, Kans.	.13	.13	July 1937	—	3.8	.080	—	—	—	SCS
	do	do	do	—	—	July 1967	30	3.0	.062	*60	.23	300	
45-50	Sierman	do	do	.23	.23	June 1937	—	5.3	.062	—	—	—	SCS
	do	do	do	—	—	July 1967	30	4.5	.052	*60	.13	170	
45-51	Hubert	do	Longton, Kans.	.41	.40	Oct. 1948	—	16.38	.11	—	—	—	SCS
	do	do	do	—	—	Aug. 1967	19	15.0	.098	*60	.18	235	
45-52	Meritt Pond	Big Creek	Burlington, Kans.	.25	.25	— 1939	—	8.09	.09	—	—	—	SCS
	do	do	do	—	—	Aug. 1967	28	4.17	.045	*60	.56	731	
45-53	Big Wewoka Site No. 36	Big Wewoka	Holdenville, Okla.	2.26	2.20	June 1960	—	690.3	.95	—	—	—	SCS
	do	do	do	—	—	Aug. 1969	9.1	679.0	.94	62	.56	762	
45-54	Cane Creek Site No. 11	Cane Creek	Okmulgee, Okla.	8.96	8.89	Nov. 1965	—	2,983.6	.96	—	—	—	SCS
	do	do	do	—	—	Sept. 1969	3.85	2,954.7	.95	55	.85	1,018	
45-55	Big Wewoka Site No. 17	Big Wewoka	Wewoka, Okla.	2.16	2.11	Mar. 1963	—	691.3	1.36	—	—	—	SCS
	do	do	do	—	—	Sept. 1968	5.53	680.7	1.34	60	.91	1,189	

ARKANSAS RIVER BASIN (GARDEN CITY TO TULSA)

Middle Canadian, Lower Cimarron, and Salt Fork River Basins

46-1b	Fort Supply Lake	Wolf Creek	Fort Supply, Okla.	1,735	1,405	Jan. 1943	—	107,340	1.27	—	—	—	CE
	do	do	do	—	—	June 1949	6.42	106,150	1.26	57.8	.125	157	
	do	do	do	—	—	June 1958	9.0	101,750	1.21	67.0	.329	498	
	do	do	do	—	—	Dec. 1969	11.50	100,720	1.66	65.3	.061	74	

1/ 22,241 sq. mi. is probably noncontributing.
2/ Based on inflow from net sediment contributing area.
3/ Original topographic survey.
4/ From "A" reconnaissance investigation, Shawnee Lake, by Victor H. Jones, Geologist, SCS.
5/ Present ac.-ft. as shown by the two surveys.
6/ Lake drained and dam raised 1937. Original capacity at 1938 crest.
7/ Dam raised 11 feet Mar. 1946; all values based on present elevation.
8/ Dam broke Apr. 1945; rebuilt Mar. 1946; this period not included.
9/ Includes 0.03 ac.-ft. above crest deposits.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

ARKANSAS RIVER BASIN (GARDEN CITY TO TULSA)
Middle Canadian, Lower Cimarron, and Salt Fork River Basins (Continued)

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
46-2a	Great Salt Plains	Salt Fk. of Arkansas River	Jet, Okla.	3,200	3,156	June 1941	—	308,000	1.067	—	—	—	CE
	do	do	do	—	—	Dec. 1949	8.5	292,000	1.011	48.6	0.986	620	
	do	do	do	—	—	Apr. 1961	11.4	280,200	.971	58.9	.336	529	
46-3	Boomer Lake	Boomer Creek	Stillwater, Okla.	9.13	8.67	Mar. 1925	—	2,812	—	—	—	—	SCS
	do	do	do	—	—	June 1935	10.25	2,641	—	*60	1.93	2,522	
46-4	Bennington's Lake	Chikaskia River	Rago, Kans.	1.42	1.40	Aug. 1929	—	75	—	—	—	—	SCS
	do	do	do	—	—	Oct. 1940	11.2	33	—	69.47	2.68	4,055	
46-5	Meade County State Lake (Lake Larrabee).	Stump Arroyo	Meade, Kans.	18.00	17.84	June 1928	—	891	—	—	—	—	SCS
	do	do	do	—	—	Apr. 1937	8.8	819	—	*60	.459	600	
46-6	Lake Medicine	Medicine Lodge Creek	Medicine Lodge, Kans.	1.89	1.84	Aug. 1929	—	365	—	—	—	—	SCS
	do	do	do	—	—	Oct. 1940	11.2	295	—	98.52	3.40	7,296	
46-7	Santa Fe	Indianola Creek	Augusta, Kans.	37.93	37.55	Oct. 1928	—	1,741	—	—	—	—	SCS
	do	do	do	—	—	May 1937	8.6	1,595	—	58.1	.45	569	
46-8	Guthrie	Trib. of Cottonwood Creek	Guthrie, Okla.	13.30	12.95	Oct. 1920	—	3,064	—	—	—	—	SCS
	do	do	do	—	—	May 1935	14.5	2,608	—	*60	2.42	3,162	
46-9	Gelmer's Stock Pond	Bois d'Arc Creek	Blackwell, Okla.	.31	.31	Oct. 1939	—	3.74	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1940	1.0	3.43	—	*65	1.0	1,416	
46-10	Harris Stock Pond	do	do	.22	.21	Apr. 1938	—	12.63	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1940	2.5	12.37	—	*65	.495	701	
46-11	Lake Eldorado	Satchel Creek	Eldorado, Kans.	35.1	34.3	— 1928	—	3,213	—	—	—	—	SCS
	do	do	do	—	—	Apr. 1937	9	3,082	—	66	.426	612	
46-12a	Lake Fryer	Wolf Creek	Perryton, Tex.	108	108	— 1939	—	844	.372	—	—	—	SCS
	do	do	do	—	—	May 1947	7.4	717	.316	—	.16	—	
46-13b	Canton Lake	North Canadian River	Canton, Okla.	12,483	1/6,081	July 1947	—	401,500	2.15	—	—	—	CE
	do	do	do	—	—	May 1953	5.83	390,800	2.10	70.9	.302	4,664	
	do	do	do	—	—	Oct. 1959	6.42	385,900	2.07	56.2	.125	65.3	
	do	do	do	—	—	Sept. 1966	6.92	383,300	2.06	56.1	.063	76.4	
46-14	Guthrie Municipal Reservoir No. 2.	Cottonwood Creek	Guthrie, Okla.	11.63	11.29	May 1969	—	3,192.68	1.391	—	—	—	SCS
	do	do	do	—	—	Sept. 1960	11.40	2,965.06	1.283	95.66	1.92	4,000.27	
46-15a	Lake Carl Blackwell	Stillwater Creek	Stillwater, Okla.	75.35	70.53	Mar. 1938	—	58,326	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1958	20.50	54,556	—	*60	2.61	3,410.75	
46-16	Lies Pond	Unnamed	Andale, Kans.	3.36	3.36	June 1942	—	20.44	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1956	14	9.62	—	96.89	2/.28	3/578.68	
46-17	Longton	do	Cimarron, Kans.	.55	.54	July 1935	—	3/51.10	—	—	—	—	SCS
	do	do	do	—	—	Aug. 1955	20	50.28	—	85.7	.07	134	
46-18	Hughes No. 1	do	Ingalls, Kans.	.64	.63	— 1942	—	3/8.50	—	—	—	—	SCS
	do	do	do	—	—	Aug. 1955	13	5.44	—	72.07	.32	502	
46-19	Perry City Lake	Cow Creek	Perry, Okla.	16.67	16.64	June 1937	—	4,371	1.311	—	—	—	SCS
	do	do	do	—	—	May 1955	17.9	3,986	1.196	83.5	1.33	2,419	
46-20	Pottesinger Pond	Unnamed	Andale, Kans.	.9	.9	Aug. 1949	—	14.87	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1956	7	8.97	—	103.4	4/1.0	4/2,252	
46-21	O. F. Scott Ponds	Trib. of So. Canadian R.	Union City, Okla.	1.80	1.79	May 1941	—	26.3	.085	—	—	—	SCS
	do	do	do	—	—	May 1955	14	20.7	.067	65.4	.22	313	
46-22	Tom Hill Pond	Trib. of Bridge Creek	Tuttle, Okla.	.530	.526	July 1943	—	13.38	.147	—	—	—	SCS
	do	do	do	—	—	May 1955	11.8	6.43	.071	—	1.12	—	
46-23	Michlaus	Trib. Slough Creek	Haven, Kans.	2.29	2.27	May 1932	—	46.14	.15	—	—	—	SCS
	do	do	do	—	—	May 1967	35	5/13.6	.04	70	.22	335	
46-24	Thayer	Trib. Slate Creek	Wellington, Kans.	.14	.14	— 1958	—	4.09	.16	—	—	—	SCS
	do	do	do	—	—	Aug. 1967	9	3.38	.13	*60	.57	745	
46-25	Koulouris	Trib. Sand Creek	Newton, Kans.	.24	.24	Oct. 1948	—	9.64	.219	—	—	—	SCS
	do	do	do	—	—	Sept. 1968	19.92	6.95	.158	*60	.58	758	
46-26	Adams	Trib. Bluff Creek	Anthony, Kans.	.24	.23	Sept. 1934	—	13.50	.56	—	—	—	SCS
	do	do	do	—	—	Aug. 1968	33.92	5.76	.24	*60	1.00	1,307	
46-27	Smith - Case	Trib. Little Ark. River	Little River, Kans.	.56	.56	Dec. 1958	—	66.89	1.86	—	—	—	SCS
	do	do	do	—	—	Aug. 1968	9.75	62.29	1.73	*60	.84	1,098	
46-28	Workman	Trib. Medicine Lodge River	Medicine Lodge, Kans.	.59	.59	June 1936	—	10.0	.18	—	—	—	SCS
	do	do	do	—	—	Aug. 1968	32.17	1.79	.03	*60	.44	575	
46-29	Gant	do	do	.30	.30	Mar. 1955	—	25.26	.87	—	—	—	SCS
	do	do	do	—	—	Aug. 1968	13.42	20.29	.70	*60	1.23	1,607	
46-30	Schupbach	do	Kiowa, Kans.	.34	.34	June 1956	—	26.73	.81	—	—	—	SCS
	do	do	do	—	—	Aug. 1968	12.17	21.91	.66	*60	1.18	1,542	
46-31	Barrett	Wild Creek	Anthony, Kans.	.67	.67	Spring 1957	—	39.92	.59	—	—	—	SCS
	do	do	do	—	—	July 1968	11.25	32.24	.47	*60	1.01	1,320	
46-32	Miller	Beaver Creek	Walton, Kans.	1.20	1.20	Aug. 1959	—	6.08	.028	—	—	—	SCS
	do	do	do	—	—	July 1968	9	2.87	.013	*60	.30	392	

46-33	Roupp	Trib. Sand Creek	Hesston, Kans.	.49	.49	June	1950	—	5.47	.066	—	—	—	SCS
	do	do	do	—	—	Sept.	1968	18.12	3.33	.037	*60	.24	314	
46-34	Mease	Trib. Medicine Lodge River	Medicine Lodge, Kans.	1.73	1.71	Feb.	1961	—	108.3	.65	—	—	—	SCS
	do	do	do	—	—	Aug.	1968	7.5	105.19	.63	*60	.24	314	
46-35	Berger Pond	Trib. Pawnee River	Jetmore, Kans.	3.0	3.0	Mar.	1952	—	18.55	.29	—	—	—	SCS
	do	do	do	—	—	Aug.	1967	15.5	12.95	.20	*60	.12	156	
46-36	Paris	Trib. Wet Walnut Creek	Dighton, Kans.	3.6	2.2	Aug.	1940	—	41.69	.87	—	—	—	SCS
	do	do	do	—	—	Aug.	1967	27	36.66	.76	*60	.09	117	
46-37	Siebert	Trib. Big Sandy	Meade, Kans.	.23	.23	Jan.	1952	—	8.0	1.6	—	—	—	SCS
	do	do	do	—	—	June	1966	14.5	7.15	1.5	80	.26	453	
46-38	Hoop	Trib. Crooked Creek	do	1.07	.90	May	1961	—	4.0	.19	—	—	—	SCS
	do	do	do	—	—	June	1966	5.2	3.76	.19	80	.06	104	
46-39	Harris	do	do	7.57	4.75	Oct.	1962	—	161.4	1.1	—	—	—	SCS
	do	do	do	—	—	June	1966	3.7	160.8	1.1	*70	.04	61	
46-40	Whitford	Trib. Hackberry Creek	Ashland, Kans.	1.98	1.97	May	1947	—	14.2	.22	—	—	—	SCS
	do	do	do	—	—	July	1966	19.2	8.2	.13	75	.15	245	
46-41	Davis	Pawnee	Kalvesta, Kans.	2.40	2.37	Jan.	1951	—	104.82	3.28	—	—	—	SCS
	do	do	do	—	—	Sept.	1967	16.75	97.94	3.06	*60	.17	222	
46-42	Lippoldt	Trib. Pawnee River	Jetmore, Kans.	1.83	1.82	Dec.	1952	—	6/30.5	.78	—	—	—	SCS
	do	do	do	—	—	Aug.	1967	14.75	27.74	.71	*60	.10	131	
46-43	Sebes Pond	do	do	1.20	1.20	July	1956	—	5.6	.22	—	—	—	SCS
	do	do	do	—	—	Aug.	1967	11.13	4.34	.17	*60	.09	117	
46-44	Fry	Trib. Ark. River	Little River, Kans.	1.07	1.07	Aug.	1949	—	4.13	.06	—	—	—	SCS
	do	do	do	—	—	Aug.	1968	19	1.37	.02	*60	.14	183	
46-45	Barber	do	Raymond, Kans.	.57	.57	June	1962	—	21.50	.60	—	—	—	SCS
	do	do	do	—	—	June	1968	6.2	17.97	.50	*70	1.0	1,525	
46-46	Coberly	Trib. Buffalo Creek	Dighton, Kans.	2.1	2.1	Aug.	1946	—	18.0	.65	—	—	—	SCS
	do	do	do	—	—	Aug.	1967	21	12.93	.48	*60	.11	143	
46-47	Smith Pond	Trib. Pawnee River	Jetmore, Kans.	.30	.30	Dec.	1951	—	5.93	.93	—	—	—	SCS
	do	do	do	—	—	Aug.	1967	15.75	3.56	.59	*60	.50	653	
46-48	Saltzman	Trib. Walnut Creek	Rush Center, Kans.	1.63	1.61	Oct.	1958	—	26.07	.43	—	—	—	SCS
	do	do	do	—	—	Aug.	1967	8.8	22.23	.37	*60	.27	352	
46-49	Rayl	Trib. Slate Creek	Wellington, Kans.	.56	.56	—	1949	—	5.44	.05	—	—	—	SCS
	do	do	do	—	—	Aug.	1967	18	2.96	.03	*60	.25	326	
46-50	Lange	do	Conway Springs, Kans.	.31	.31	—	1956	—	8.11	.14	—	—	—	SCS
	do	do	do	—	—	Aug.	1967	11	5.37	.09	*60	.81	1,058	
46-51	Brodie	Spring Creek	Ashland, Kans.	2.17	2.0	June	1951	—	31.2	.36	—	—	—	SCS
	do	do	do	—	—	July	1966	15.1	29.0	.33	*70	.07	107	
46-52	Glen	Trib. Little Sandy Creek	do	.95	.93	Nov.	1958	—	57.1	1.88	—	—	—	SCS
	do	do	do	—	—	July	1966	7.75	54.5	1.79	*60	.35	457	
46-53	Isaac	Trib. Big Sandy	Meade, Kans.	.82	.52	Jan.	1952	—	6.65	.38	—	—	—	SCS
	do	do	do	—	—	June	1966	14.5	4.8	.27	75	.25	408	
46-54	Fox	Trib. Bluff Creek	Ashland, Kans.	2.18	2.17	July	1954	—	10.0	.11	—	—	—	SCS
	do	do	do	—	—	July	1966	12	7.3	.08	*65	.11	156	

ARKANSAS RIVER BASIN (LAMAR TO GARDEN CITY)
Upper Cimarron and Upper Canadian River Basins

47-1b	Conchas Reservoir	Canadian and Conchas	Conchas Dam, N. Mex.	7,409	6,976	Jan.	1939	—	601,112	3.13	—	—	—	CE
	do	do	do	—	—	May	1940	1.4	599,712	3.12	75.7	—	—	
	do	do	do	—	—	June	1942	2.1	585,112	3.04	75.7	.996	1,643	
	do	do	do	—	—	Nov.	1942	.4	581,112	3.02	75.7	—	—	
	do	do	do	—	—	Oct.	1944	1.9	576,756	3.00	75.7	.328	541	
	do	do	do	—	—	Feb.	1949	4.3	566,163	2.95	*75.7	.353	581	
	do	do	do	—	—	Oct.	1963	14.7	550,799	2.87	*75.7	.150	247	
	do	do	do	—	—	Oct.	1970	6.92	528,951	2.75	75.7	.453	746	
47-2	Reservoir No. 2	(Offstream)	Maxwell, N. Mex.	—	—	—	1912	—	3,010	—	—	—	—	BR
	do	do	do	—	—	—	1946	34	1,836	—	—	—	—	
47-3	Reservoir No. 7 and 8	do	do	—	—	—	1912	—	7/1,064	—	—	—	—	BR
	do	do	do	—	—	—	1946	34	2,063	—	—	—	—	
47-4	Reservoir No. 11	do	do	—	—	—	1912	—	8/1,006	—	—	—	—	BR
	do	do	do	—	—	—	1946	34	637	—	—	—	—	
47-5	Reservoir No. 12	do	do	—	—	—	1912	—	9/2,260	—	—	—	—	BR
	do	do	do	—	—	—	1946	34	1,822	—	—	—	—	
47-6	Reservoir No. 13	do	do	—	—	—	1912	—	10/2,236	—	—	—	—	BR
	do	do	do	—	—	—	1946	34	1,628	—	—	—	—	
47-7	Reservoir No. 14	do	do	—	—	—	1912	—	11/816	—	—	—	—	BR
	do	do	do	—	—	—	1946	34	680	—	—	—	—	
47-8	Baleo	Cimarron	Liberal, Kans.	.36	.36	Mar.	1962	—	9.16	1.9	—	—	—	SCS
	do	do	do	—	—	June	1966	4.25	8.98	1.9	*65	.12	170	

[1] Excludes 4,642 sq. mi. of watershed not contributed to runoff, 1,735 sq. mi. above Fort Supply Dam, and 25 sq. mi. surface area of Canton Reservoir.

[2] Includes 2.33 ac.-ft. above crest deposits.

[3] Water supply pool capacity. Reservoir has greater capacity at spillway crest elevation.

[4] Includes 0.39 ac.-ft. above crest deposits.

[5] Spillway eroded 2 ft.

[6] Capacity based on surface area x 1/3 deepest fill.

[7] During the period 1912-46, a total of 2,435-ac.-ft. capacity was added by reservoir enlargement.

[8] During the period 1912-46, a total of 126-ac.-ft. capacity was added by reservoir enlargement.

[9] During the period 1912-46, a total of 672-ac.-ft. capacity was added by reservoir enlargement.

[10] During the period 1912-46, a total of 490-ac.-ft. capacity was added by reservoir enlargement.

[11] During the period 1912-46, a total of 264-ac.-ft. capacity was added by reservoir enlargement.

* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE -FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
ARKANSAS RIVER BASIN (LAMAR TO GARDEN CITY) Upper Cimarron and Upper Canadian River Basins (Continued)													
47-9	Gregory	Trib. Ark. River	Syracuse, Kans.	6.5	6.5	Apr. 1951	—	62.2	1.79	—	—	—	SCS
	do	do	do	—	—	June 1967	16.5	50.83	1.50	70	0.11	167	
47-10	Lahey	Trib. Cimarron	Moscow, Kans.	2.78	2.75	Oct. 1960	—	135.6	4.6	—	—	—	SCS
	do	do	do	—	—	June 1966	5.7	133.8	4.5	*50	.11	120	
47-11	Hartshorn	Trib. Little Bear Creek	Syracuse, Kans.	1.90	1.89	Mar. 1950	—	14.6	.72	—	—	—	SCS
	do	do	do	—	—	June 1966	16.25	8.75	.43	*75	.19	310	
47-12	Christian	Trib. Cimarron	Ulysses, Kans.	1.06	1.06	June 1961	—	4.5	.40	—	—	—	SCS
	do	do	do	—	—	June 1966	5	3.75	.33	*75	.14	229	
47-13	Downing	do	Moscow, Kans.	.47	.46	Aug. 1961	—	31.8	6.4	—	—	—	SCS
	do	do	do	—	—	June 1966	4.8	31.2	6.2	65	.28	396	
47-14	Amerine	Trib. Little Bear Creek	Syracuse. Kans.	3.42	3.40	Mar. 1953	—	128	3.5	—	—	—	SCS
	do	do	do	—	—	June 1966	13.3	119	3.3	*75	.20	327	
47-15	Roy Kurz	Shirley	do	8.91	8.91	Apr. 1951	—	92.7	1.97	—	—	—	SCS
	do	do	do	—	—	June 1967	16.5	78.2	1.65	*70	.10	152	
RIO GRANDE BASIN (ABOVE ESPANOLA) AND ARKANSAS RIVER BASIN													
48-1d	John Martin Reservoir (formerly Caddoa Res. Proj.).	Arkansas River	Hasty, Colo.	1/18,915	18,102	Apr. 1942	—	701,775	2.47	—	—	—	CE
	do	do	do	—	—	July 1942	0.3	690,345	2.43	*75.7	—	—	
	do	do	do	—	—	Dec. 1943	1.4	688,550	2.43	*75.7	.075	124	
	do	do	do	—	—	Sept. 1944	0.7	683,257	2.41	*75.7	.443	730	
	do	do	do	—	—	May 1948	3.7	675,097	2.38	*75.7	.129	213	
	do	do	do	—	—	Oct. 1951	3.4	662,870	2.34	*75.7	.210	346	
	do	do	do	—	—	Aug. 1957	5.7	645,512	2.28	*75.7	.178	293	
	do	do	do	—	—	Mar. 1962	4.6	642,390	2.26	*75.7	.0398	65.6	
	do	do	do	—	—	Sept. 1966	4.5	631,121	2.22	*75.7	.147	242	
	do	do	do	—	—	Aug. 1968	1.9	618,660	2.18	*75.7	.362	597	
48-2	Brown Reservoir No. 1	Van Bremer Arroyo	Trinidad, Colo.	74.6	74.4	— 1900	—	758	—	—	—	—	SCS
	do	do	do	—	—	Nov. 1939	39	82	—	89.07	.233	452	
48-3	Muddy Creek	Muddy Creek, Johnny Creek	Caddoa, Colo.	154.2	152.4	Fall 1919	—	16,918	—	—	—	—	SCS
	do	do	do	—	—	Dec. 1939	20	15,287	—	75.25	.535	877	
48-4	Horse Creek 2/	Horse Creek	Ordway, Colo.	52.01	47.44	Nov. 1900	—	36,203	—	—	—	—	SCS
	do	do	do	—	—	Apr. 1940	39.4	30,738	—	68.36	3/.241	4/1,515	
48-5	Teller	Turkey Creek	Pueblo, Colo.	78.8	78.5	Mar. 1911	—	4,005	—	—	—	—	SCS
	do	do	do	—	—	Feb. 1940	28.9	2,463	—	75.4	.680	1,117	
48-6	Cucharas	Cucharas River	Walsenburg, Colo.	608	608	— 1912	—	38,274	—	—	—	—	BR
	do	do	do	—	—	— 1937	25	25,020	—	—	.87	—	
	do	do	do	—	—	— 1939	2	23,040	—	—	1.63	—	
48-7	S-1 Big Sandy Creek Watershed	Big Sandy Creek	Peyton, Colo.	5.4	5.3	May 1962	—	326	—	—	—	—	SCS
	do	do	do	—	—	July 1965	3	302	—	*80	1.5	2,613	
48-8	Abiquiu Reservoir	Rio Chama	Abiquiu, N. Mex.	2,146	2,127	Apr. 1963	—	579,039	1.80	—	—	—	CE
	do	do	do	—	—	Oct. 1967	4.58	572,695	1.78	*75.7	.65	1,072	
48-9	Hardesty Reservoir	Hungerford Hollow	Fowler, Colo.	13.48	13.34	— 1908	—	563	*3.9	—	—	—	SCS
	do	do	do	—	—	Mar. 1968	60	527	*3.7	—	5/.045	—	
RED RIVER BASIN (DENISON TO GRAND ECORE) Little and Sulphur River Basins													
49-1a	Lake Crook	Pine Creek	Paris, Tex.	51.6	49.6	Feb. 1923	—	11,487	.418	—	—	—	SCS
	do	do	do	—	—	Mar. 1936	13.1	10,755	.391	—	1.13	—	
	do	do	do	—	—	July 1956	19.7	9,964	.362	36.4	.81	642	
49-2a	Lake Gibbons	Trib. of Pine Creek	do	1.46	1.26	— 1900	—	6/1,394	1.792	—	—	—	SCS
	do	do	do	—	—	Mar. 1936	36	1,324	1.702	—	1.54	—	
	do	do	do	—	—	July 1956	20.3	1,285	1.652	31.5	1.52	1,063	
49-3	Nashville	Mine Creek	Nashville, Ark.	10.4	10.3	Mar. 1931	—	181	—	—	—	—	SCS
	do	do	do	—	—	July 1941	10.3	128	—	—	.50	—	
49-4	Jenkins Pond	Trib. of Lewis Creek	Mansfield, La.	.35	.32	Sept. 1925	—	157	.561	—	—	—	SCS
	do	do	do	—	—	Sept. 1954	29	150	.536	68.6	.78	1,165	
49-5	Gordon Country Club Lake	Ands Creek	Paris, Tex.	1.41	1.30	— 1898	—	558	.742	—	—	—	SCS
	do	do	do	—	—	June 1959	61.0	430	.572	33.14	1.62	1,169	
49-6	Bayou Bodcau	Bayou Bodcau River	Cotton Valley, La.	683	613	Nov. 1949	—	967,900	1.516	—	—	—	CE
	do	do	do	—	—	May 1961	11.5	967,900	1.516	—	—	—	

49-7a	Lake Texarkana	Sulphur River	Texarkana, Tex.	3,400	3,213	Sept. 1954	—	2,654,300	.13	—	—	—	CE
	do	do	do	—	—	Aug. 1958	3.92	2,654,300	.13	—	—	—	
	do	do	do	—	—	July 1970	12	2,654,300	.13	—	—	—	
49-8	Wallace Lake	Cypress Bayou	Shreveport, La.	266	251	Dec. 1946	—	96,100	.405	—	—	—	CE
	do	do	do	—	—	Oct. 1951	4.8	96,100	.405	—	—	—	
	do	do	do	—	—	Apr. 1961	9.5	96,100	.405	—	—	—	
49-9	Century Lake	Sulphur River	Sulphur Springs, Tex.	52.50	51.46	— 1953	—	3,495.2	.125	—	—	—	SCS
	do	do	do	—	—	Sept. 1962	9	2,787.9	.100	*85	1.53	2,832	
			RED RIVER BASIN (ABOVE DENISON)										
90-1	Ardmore Club Lake	Caddo Creek	Ardmore, Okla.	4.15	3.91	Dec. 1922	—	1,797	—	—	—	—	SCS
	do	do	do	—	—	June 1938	15.5	1,644	—	40.7	2.52	2,234	
90-2	Byars Club Lake	Unnamed Stream	Byars, Okla.	2.66	2.55	— 1904	—	507	—	—	—	—	SCS
	do	do	do	—	—	— 1950	46	371	—	—	1.16	—	
90-3	Carter Lake	Big Glasses Creek	Madill, Okla.	1.81	1.70	— 1936	—	865	—	—	—	—	SCS
	do	do	do	—	—	Nov. 1949	13	837	—	—	1.26	—	
90-4	J. J. Harrison Lake	Trib. of Washita River	Lindsay, Okla.	.88	.81	Feb. 1932	—	349	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1950	18	282	—	—	4.59	—	
90-5	C. W. Lester Farm Pond No. 1	Trib. of Broken Leg Creek	Cheyenne, Okla.	2.04	2.03	— 1945	—	42.4	—	—	—	—	SCS
	do	do	do	—	—	June 1949	4	22.8	—	—	2.41	—	
90-6	C. W. Lester Farm Pond No. 2	do	do	.64	.63	June 1944	—	16.3	—	—	—	—	SCS
	do	do	do	—	—	June 1949	5.0	13.1	—	—	1.02	—	
90-7	Santa Rosa Lake	Beaver Creek	Vernon, Tex.	336	334	Oct. 1929	—	15,755	—	—	—	—	SCS
	do	do	do	—	—	Jan. 1948	18.2	11,568	—	—	.489	—	
90-8	Lake Duncan	Fitzpatrick Creek	Duncan, Okla.	11.0	10.4	Oct. 1937	—	6,291	—	—	—	—	SCS
	do	do	do	—	—	Aug. 1950	12.8	5,783	—	63.4	3.82	5,275	
90-9	Lake Clinton	Turkey Creek	Canute, Okla.	23.6	23.1	Dec. 1930	—	4,415	—	—	—	—	SCS
	do	do	do	—	—	June 1938	7.4	3,981	—	63.36	2.54	3,505	
	do	do	do	—	—	Nov. 1950	12.4	3,343	—	65.9	2.23	3,201	
90-10	Bellevue	Clay Creek	Bellevue, Tex.	*1.5	1.44	— 1888	—	131	—	—	—	—	SCS
	do	do	do	—	—	May 1938	50	85	—	—	.64	—	
90-11e	Altus Reservoir	North Fork Red River	Altus, Okla.	2,515	2,104	Dec. 1940 7/	—	192,842	—	—	—	—	BR
	do	do	do	—	—	June 1948	7.5	185,035	—	52.3	.495	564	
	do	do	do	—	—	July 1953	5.1	178,610	1.74	66.9	.599	1,104	
	do	do	do	—	—	Apr. 1967	13.8	168,117	1.61	70.2	.361	588	
90-12b	Lake Texoma (Denison Dam)	Red River	Denison, Tex.	39,719	28,925	July 1942	—	5,859,000	—	—	—	—	CE
	do	do	do	—	—	Oct. 1948	6.2	5,718,000	—	52.1	.785	891	
	do	do	do	—	—	June 1954	5.7	5,553,000	—	57.9	1.00	1,261	
	do	do	do	—	—	Mar. 1962	7.8	5,392,900	—	62.5	.709	965	
90-13	Barbour Pond	Trib. of Little Washita R.	Chickasha, Okla.	.68	.67	Sept. 1935	—	59.9	.512	—	—	—	SCS
	do	do	do	—	—	May 1955	19.7	41.8	.357	*75	1.37	2,238	
90-14a	Cavalry Creek Site No. 1	Washita	Cordell, Okla.	2.19	2.15	July 1948	—	505.4	3.05	—	—	—	SCS
	do	do	do	—	—	Sept. 1959	11.15	476.6	2.87	74.5	1.20	1,946	
	do	do	do	—	—	Aug. 1964	4.89	471.8	2.84	74.5	.46	746	
	do	do	do	—	—	June 1969	4.85	458.8	2.77	*74	1.25	1,993	
90-15a	Chigley Sandy Site No. 5	Chigley Sandy	Wynnewood, Okla.	.81	.79	June 1955	—	263.67	1.410	—	—	—	SCS
	do	do	do	—	—	Aug. 1959	4.12	258.74	1.384	60.3	1.52	1,996	
	do	do	do	—	—	Oct. 1963	4.15	248.20	1.327	60.3	3.22	4,229	
90-16	Coit Pond No. 1	Unnamed	Clinton, Okla.	.39	.38	Mar. 1947	—	25.69	.706	—	—	—	SCS
	do	do	do	—	—	July 1957	10.28	21.53	.591	61.49	1.07	1,433	
90-17	Dean Pond No. 1	Trib. of Nine Mile Creek	Cheyenne, Okla.	.0359	.0354	Spring 1944	—	1.078	.469	—	—	—	SCS
	do	do	do	—	—	Mar. 1955	11	.807	.351	91	.69	1,368	
90-18	Dean Pond No. 2	do	do	.164	.161	Spring 1941	—	15.8	1.505	—	—	—	SCS
	do	do	do	—	—	Mar. 1955	14	13.4	1.276	88	1.06	2,032	
90-19	George Pond	Unnamed	Canadian, Tex.	.535	.535	June 1952	—	15.197	.407	—	—	—	SCS
	do	do	do	—	—	Apr. 1958	5.796	13.463	.361	70.95	.559	863.82	
90-20	Hall Pond No. 1	do	Leedey, Okla.	.20	.20	May 1939	—	17.59	.239	—	—	—	SCS
	do	do	do	—	—	Aug. 1957	18.25	13.47	.183	*70.2	1.15	1,758.30	
90-21	Harrison No. 1	do	Canadian, Tex.	.237	.237	May 1952	—	22.163	.594	—	—	—	SCS
	do	do	do	—	—	Apr. 1958	5.93	21.706	.581	79.68	.325	564.68	
90-22	Harrison No. 2	do	do	.292	.292	Sept. 1951	—	15.655	.419	—	—	—	SCS
	do	do	do	—	—	Apr. 1958	6.59	14.242	.382	68.53	.733	1,094.06	
90-23	Lake Kemp	Wichita River	Seymour, Tex.	2,099	2,067	Oct. 1922	—	560,000	2.493	—	—	—	SCS
	do	do	do	—	—	Sept. 1958	36	461,757	2.056	—	1.32	—	
90-24a	Mill Creek No. 17	Washita	Mill Creek, Okla.	1.61	1.58	Dec. 1948	—	498.45	1.06	—	—	—	SCS
	do	do	do	—	—	July 1959	10.63	492.13	1.04	70	.37	564	
	do	do	do	—	—	Aug. 1966	7.06	487.11	1.03	*75	.45	816	
90-25	Moncrief Farm Pond	Trib. of Sandy Creek	Wynnewood, Okla.	.031	.029	Aug. 1935	—	8.710	—	—	—	—	SCS
	do	do	do	—	—	Nov. 1957	22.22	7.968	—	*60	1.15	1,902.82	

1/ Drainage area adjusted to conform with U.S.G.S. published drainage area.
2/ Off channel reservoir.
3/ Excludes water and sediment diverted from Arkansas River and Horse Creek.
4/ Per 100 ac.-ft. of water diverted from Ark. River and Horse Cr. Total of 564,918 ac.-ft. diverted to reservoir between 1910 and 1939.
5/ Little deposition due to dam washout, 1930-47. True sediment accumulation rate for 43-yr period was about 0.063 ac.-ft./mi^2-yr.
6/ Spillway crest was lowered 3 ft. in 1932; capacities are based on present elevation.
7/ Date of original survey for new dam over deposits placed behind old dam.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
	RED RIVER BASIN (ABOVE DENISON) (Continued)												
50-26a	Cobb Creek No. 3	Washita	Weatherford, Okla.	8.28	8.18	Jan. 1957	—	2,401.1	2.72	—	—	—	SCS
	do	do	do	—	—	Aug. 1960	3.58	2,303.3	2.61	90	3.34	6,547	
	do	do	do	—	—	Oct. 1964	4.14	2,277.3	2.58	90	.77	1,509	
	do	do	do	—	—	Aug. 1970	5.81	2,218.1	2.51	*91	1.21	2,563	
50-27b	Sandstone Site No. 1	do	Hammon, Okla.	5.33	5.22	May 1951	—	1,591.29	4.67	—	—	—	SCS
	do	do	do	—	—	Apr. 1956	4.96	1,453.69	4.26	61.45	5.31	7,111	
	do	do	do	—	—	May 1960	4.06	1,380.47	4.05	83.11	3.46	6,263	
	do	do	do	—	—	Aug. 1965	5.02	1,344.75	3.96	83.11	1.36	2,462	
	do	do	do	—	—	July 1970	4.92	1,338.82	3.93	*83	.23	360	
50-28b	Sandstone Site No. 3	do	Elk City, Okla.	.62	.61	Apr. 1951	—	157.68	4.38	—	—	—	SCS
	do	do	do	—	—	Oct. 1956	5.47	148.40	4.12	70.20	2.79	4,266	
	do	do	do	—	—	Oct. 1961	5.00	139.98	3.89	77.87	2.75	4,672	
	do	do	do	—	—	July 1966	4.80	125.60	3.49	*77	4.92	8,184	
50-29a	Sandstone Site No. 5	do	do	3.89	3.84	Mar. 1951	—	1,273.21	4.45	—	—	—	SCS
	do	do	do	—	—	July 1957	6.39	1,217.58	4.26	95.7	2.27	4,732	
	do	do	do	—	—	Sept. 1961	4.19	1,152.23	4.03	—	4.06	5,901	
	do	do	do	—	—	July 1966	4.81	1,127.06	3.96	*80	1.36	1,980	
50-30b	Sandstone Site No. 6	Sandstone	do	6.46	5.24	Apr. 1951	—	1,956.05	4.12	—	—	—	SCS
	do	do	do	—	—	Oct. 1962	11.53	1,813.2	3.82	70.7	2.36	3,634	
	do	do	do	—	—	Sept. 1967	4.96	1,806.22	3.80	*72	.27	529	
50-31a	Sandstone Site No. 9	do	do	3.50	3.46	Mar. 1951	—	1,275.09	4.96	—	—	—	SCS
	do	do	do	—	—	Oct. 1957	6.58	1,217.59	4.72	75.78	2.52	4,160	
	do	do	do	—	—	Oct. 1962	5.0	1,178.72	4.57	79.4	2.20	4,148	
	do	do	do	—	—	Sept. 1967	4.93	1,175.25	4.56	*80	.20	436	
50-32b	Sandstone Site No. 10A	Washita	do	2.87	2.80	Apr. 1951	—	1,046.02	6.23	—	—	—	SCS
	do	do	do	—	—	Oct. 1956	5.50	1,018.8	6.06	79.3	1.77	3,057	
	do	do	do	—	—	Sept. 1961	4.90	943.9	5.62	82.8	5.46	9,996	
	do	do	do	—	—	July 1966	4.80	888.0	5.29	*75	4.16	5,627	
50-33b	Sandstone Site No. 14	do	do	1.02	1.00	Sept. 1951	—	315.15	4.85	—	—	—	SCS
	do	do	do	—	—	May 1960	8.68	288.09	4.43	93.08	3.12	6,320	
	do	do	do	—	—	Aug. 1965	5.25	286.79	4.40	68.1	.25	371	
	do	do	do	—	—	July 1970	4.98	275.82	4.24	*75	2.20	3,611	
50-34b	Sandstone Site No. 16	do	do	11.47	11.28	Aug. 1952	—	4,463.43	7.29	—	—	—	SCS
	do	do	do	—	—	July 1957	4.92	4,296.49	7.02	77.96	3.06	5,161	
	do	do	do	—	—	Sept. 1961	4.21	4,164.87	6.81	74.85	2.73	4,209	
	do	do	do	—	—	Aug. 1966	4.86	4,038.34	6.60	*76	2.31	3,874	
50-35b	Sandstone Site No. 16A	do	Cheyenne, Okla.	8.78	5.11	Dec. 1951	—	2,065.88	4.41	—	—	—	SCS
	do	do	do	—	—	Oct. 1956	4.82	2,033.89	4.35	80.6	1.30	2,281	
	do	do	do	—	—	Sept. 1961	4.88	1,987.93	4.25	79.79	1.84	3,181	
	do	do	do	—	—	Aug. 1966	4.86	1,979.45	4.23	*77	.34	402	
50-36b	Sandstone Site No. 17	Current Creek	do	10.13	10.04	Aug. 1951	—	3,584.9	5.53	—	—	—	SCS
	do	do	do	—	—	Oct. 1956	5.13	3,465.8	5.35	70.4	2.91	3,540	
	do	do	do	—	—	Oct. 1960	4.04	3,322.4	5.13	59	3.53	5,418	
	do	do	do	—	—	Aug. 1965	4.79	3,292.7	5.08	59	.62	797	
	do	do	do	—	—	June 1970	4.91	3,276.0	5.06	*60	.34	526	
50-37a	Sandstone Site No. 22	Sandstone	do	2.25	2.20	Aug. 1951	—	970.7	6.14	—	—	—	SCS
	do	do	do	—	—	Oct. 1957	6.16	942.12	5.97	81.53	2.08	3,696	
	do	do	do	—	—	Oct. 1962	5.0	913.51	5.78	79.4	2.64	4,445	
	do	do	do	—	—	Sept. 1967	4.96	873.9	5.63	*83	3.63	7,036	
50-38	Stickley Pond No. 4	Canadian River	Canadian, Tex.	.171	.171	Jan. 1947	—	15.796	.329	—	—	—	SCS
	do	do	do	—	—	Apr. 1958	11.266	13.963	.290	77.65	.959	1,621.88	
50-39	Stickley Pond No. 6	do	do	.162	.162	Oct. 1945	—	21.035	.438	—	—	—	SCS
	do	do	do	—	—	Apr. 1958	12.564	16.627	.346	85.34	2.167	4,027.81	
50-40	Wheeler Farm Pond	do	do	.205	.205	Dec. 1944	—	7.353	.197	—	—	—	SCS
	do	do	do	—	—	Apr. 1958	13.33	6.673	.179	82.73	.249	448.66	
50-41b	Whiteshield Creek Site No. 4	Washita	Elk City, Okla.	.62	.60	Jan. 1949	—	223.4	4.33	—	—	—	SCS
	do	do	do	—	—	Sept. 1959	10.70	207.7	4.02	75.68	2.45	4,038.4	
	do	do	do	—	—	Aug. 1964	4.91	199.6	3.87	75.68	2.75	4,532.9	
	do	do	do	—	—	June 1969	4.85	179.9	3.49	*75	6.77	10,872	
50-42b	Wildhorse Site No. 1	Wildhorse	Davis, Okla.	.97	.95	May 1949	—	242.4	1.11	—	—	—	SCS
	do	do	do	—	—	July 1959	10.25	236.2	1.08	61	.63	834.1	
	do	do	do	—	—	Oct. 1963	4.25	233.8	1.07	61	.60	794.4	
	do	do	do	—	—	July 1968	4.82	229.3	1.05	*61	.98	1,274.6	
50-43a	Wildhorse Creek Site No. 33	do	Duncan, Okla.	1.77	1.73	Mar. 1950	—	587.38	1.780	—	—	—	SCS
	do	do	do	—	—	Sept. 1960	10.50	566.24	1.716	85	1.16	2,147	
	do	do	do	—	—	Aug. 1965	4.95	548.66	1.663	76	2.05	3,306	

50-44b	Owl Creek No. 1	Owl Creek	Wayne, Okla.	.61	.59	Apr.	1949	—	206.3	.92	—	—	—	SCS
	do	do	do	—	—	Aug.	1959	10.32	200.0	.88	65.5	1.07	1,527.2	
	do	do	do	—	—	Oct.	1963	4.12	196.0	.86	65.5	2.03	3,166.6	
	do	do	do	—	—	Aug.	1968	4.84	190.3	.84	*63	1.29	1,721.1	
50-45a	Chigley Sandy Site No. 4	Washita	Wynnewood, Okla.	3.80	3.73	Aug.	1955	—	1,075.3	1.24	—	—	—	SCS
	do	do	do	—	—	Mar.	1958	2.62	1,054.8	1.21	51.61	2.10	2,361	
	do	do	do	—	—	Oct.	1963	5.52	1,018.0	1.17	51.61	1.79	2,012	
	do	do	do	—	—	July	1969	5.76	966.7	1.11	*59	2.39	3,456	
50-46	Flood Retarding Structure No. 2 Rush Cr. Watershed	Rush Creek	Rush Springs, Okla.	2.13	2.05	Aug.	1959	—	655.99	1.929	—	—	—	SCS
	do	do	do	—	—	July	1965	5.90	639.60	1.881	77.75	1.36	2,303	
50-47a	Barnitz No. 14	Barnitz	Clinton, Okla.	4.22	4.11	Nov.	1958	—	1,723.2	4.28	—	—	—	SCS
	do	do	do	—	—	Sept.	1963	4.85	1,665.9	4.08	*70	3.87	5,900	
	do	do	do	—	—	Aug.	1968	4.87	1,633.9	4.05	*70	.61	877.9	
50-48	Sugar Creek Site No. 13	Sugar Creek	Hinton, Okla.	1.99	1.93	Feb.	1964	—	532.17	2.137	—	—	—	SCS
	do	do	do	—	—	Oct.	1965	1.70	503.79	2.023	82.27	8.65	15,499	
50-49	Upper Washita River Site #25	Washita River	Canadian, Tex.	7.14	6.96	Nov.	1961	—	1,315.92	4.62	—	—	—	SCS
	do	do	do	—	—	July	1966	4.6	1,312.74	4.61	91.8	1/.10	1/199.9	
50-50	Kent Creek Watershed Site #1	Kent Creek	Quitaque, Tex.	1.52	1.46	Apr.	1964	—	622.01	15.4	—	—	—	SCS
	do	do	do	—	—	June	1966	2.1	613.83	15.2	61	1/2.67	1/3,547	
50-51	Rush Creek No. 2	Rush Creek	Rush Springs, Okla.	2.13	2.05	Aug.	1959	—	656.0	2.31	—	—	—	SCS
	do	do	do	—	—	July	1965	5.9	639.6	2.25	77.75	1.36	2,303	
	do	do	do	—	—	Sept.	1970	5.19	635.4	2.24	*77	.39	654	
50-52	Saddle Mountain No. 2	Saddle Mountain Creek	Carnegie, Okla.	3.43	3.37	Apr.	1959	—	771.3	2.11	—	—	—	SCS
	do	do	do	—	—	Apr.	1965	6	2/—	—	38	2/.47	2/389	
	do	do	do	—	—	Sept.	1970	5.26	751.4	2.05	*60	.53	734	

SABINE, NECHES, AND TRINITY RIVER BASINS

51-1	Terrell City Lake	Kings Creek	Terrell, Tex.	9.20	8.71	Oct.	1921	—	2,219	—	—	—	—	SCS
	do	do	do	—	—	Dec.	1949	28.25	1,605	—	59.2	2.49	3,211	
51-2	Lower Benton Lake	Trib. of Elm Creek	Corsicana, Tex.	.91	.82	—	1893*	—	319	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1949	56	205	—	—	2.57	—	
51-3	Burke Rock Lake	do	do	.58	.54	—	1880	—	181	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1949	69	92	—	—	2.39	—	
51-4	Lake Dallas	Elm Fk. of Trinity River	Denton, Tex.	1,174	1,157	Feb.	1928	—	180,759	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1938	10.5	167,072	—	53	1.13	1,304	
51-5	Lake Halbert	Elm Creek	Corsicana, Tex.	3/9.48	8.56	—	1921	—	8,012	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1949	28	6,657	—	67.4	5.66	8,308	
51-6	Magnolia Lake (Red Horse)	Trib. of Elm Creek	do	.59	.43	—	1885	—	756	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1949	64	659	—	—	3.53	—	
51-7	Mountain Creek	Mountain Creek	Dallas, Tex.	280	274.4	Mar.	1937	—	37,520	—	—	—	—	SCS
	do	do	do	—	—	Nov.	1946	9.7	27,100	—	—	3.91	—	
51-8b	White Rock Lake	White Rock Creek	do	99.1	97.4	Apr.	1910	—	18,158	.80	—	—	—	SCS
	do	do	do	—	—	Apr.	1935	25	14,276	.63	49	1.60	1,708	
	do	do	do	—	—	Mar.	1956	20.9	12,321	.54	35	.97	151	
	do	do	do	—	—	Oct.	1970	14.6	10,743	.47	32	1.11	505	
51-9	Grand Saline	Simmons Branch	Grand Saline, Tex.	2.12	2.02	Feb.	1925	—	531	—	—	—	—	SCS
	do	do	do	—	—	Apr.	1938	13.25	509	—	38.7	.82	691	
51-10	T & P	Town Creek	Weatherford, Tex.	6.24	6.18	—	1930	—	370	—	—	—	—	SCS
	do	do	do	—	—	Nov.	1938	8.5	328	—	61	.796	1,058	
51-11	Lake Clark	Waxahachie Creek	Ennis, Tex.	3.14	2.87	—	4/	—	5/2,085	—	—	—	—	SCS
	do	do	do	—	—	Dec.	1950	—	5/1,741	—	*50	5/2.60	2,831	
51-12	Kemp City	Cedar Creek	Kemp, Tex.	1.48	1.42	June	1926	—	376	—	—	—	—	SCS
	do	do	do	—	—	Apr.	1939	12.8	298	—	—	4.29	—	
51-13	Variety Club Boys' Ranch Lake	Unnamed trib.	Bedford, Tex.	.30	.29	June	1942	—	38	—	—	—	—	SCS
	do	do	do	—	—	Apr.	1950	7.8	33	—	—	2.21	—	
51-14	Wills Point	Mughes Creek	Wills Point, Tex.	1.83	1.75	Sept.	1914	—	396	—	—	—	—	SCS
	do	do	do	—	—	Apr.	1938	23.6	275	—	60.1	2.93	3,835	
51-15	Bridgeport	West Fork of Trinity River	Bridgeport, Tex.	1,051	1,033	—	1932	—	6/292,000	—	—	—	—	SCS
	do	do	do	—	—	Feb.	1943	10.8	283,240	—	—	.705	—	
51-16a	Eagle Mountain	do	Fort Worth, Tex.	1,875	7/809	Mar.	1934	—	6/211,000	—	—	—	—	SCS
	do	do	do	—	—	Mar.	1939	5	205,175	—	—	1.44	—	
	do	do	do	—	—	Nov.	1952	13	182,000	—	—	2.20	—	
51-17	Lake Erie	Unnamed trib.	do	1.05	1.01	—	1899	—	270	—	—	—	—	SCS
	do	do	do	—	—	Apr.	1939	40	204	—	—	1.63	—	
51-18	Mohawk City Lake	Trib. of Cedar Creek	Athens, Tex.	.36	.33	—	1926	—	295	—	—	—	—	SCS
	do	do	do	—	—	Apr.	1939	13	271	—	—	5.51	—	
51-19	Wolf Creek	Wolf Creek	Palestine, Tex.	2.55	2.50	—	1919	—	222	—	—	—	—	SCS
	do	do	do	—	—	Apr.	1939	20	199	—	—	.46	—	
51-20	Elkins Lake	East Sandy Creek	Huntsville, Tex.	3.18	3.05	—	1930	—	856	—	—	—	—	SCS
	do	do	do	—	—	July	1950	20	812	—	—	.72	—	

1/ Sediment pool only.
2/ Area above sediment pool riser was not surveyed in 1965.
3/ Excludes area above two lakes in watershed which contributes occasional flow to Lake Halbert.
4/ Lake Clark was built in 1940 downstream from "Ennis New Lake" or "Club Lake" which was built in 1895. Club Lake and a small pond now submerged by Lake Clark.
5/ Data based upon combined capacities, drainage area, and sediment volumes of all lakes in system.
6/ Water supply pool capacity. Does not include surcharge or flood storage.
7/ Excludes area above Bridgeport Reservoir which lies upstream from Eagle Mt. Reservoir.
* Estimated or assumed.

SUMMARY OF

RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
SABINE, NECHES, AND TRINITY RIVER BASINS (Continued)													
51-21	Murphy Lake	East Fork of Trinity River	Crandall, Tex.	4.09	3.98	Oct. 1922	—	397	—	—	—	—	SCS
	do	do	do	—	—	Apr. 1939	16.5	138	—	*60	3.94	5,150	
51-22	Loring Lake	Trib. of Harrison Creek	Zwolle, La.	1.05	.95	Sept. 1928	—	663	.789	—	—	—	SCS
	do	do	do	—	—	Sept. 1954	26	623	.742	85.08	1.62	3,002	
51-23	Kerens City Lake	Cow Creek	Kerens, Tex.	6.42	6.28	Sept. 1935	—	630	.237	—	—	—	SCS
	do	do	do	—	—	July 1954	18.8	554	.208	54.49	.64	759	
51-24	Variety Club Lake	Trib. of Trinity River	Bedford, Tex.	.29	.29	July 1942	—	38	*.491	—	—	—	SCS
	do	do	do	—	—	May 1950	7.8	33	*.426	—	2.21	—	
51-25a	Clear Fk. Watershed Site No. 10.	Trinity River	Weatherford, Tex.	4.30	4.25	Oct. 1955	—	1,473.49	1.90	—	—	—	SCS
	do	do	do	—	—	Apr. 1963	7.5	1,430.91	1.84	—	—	—	
	do	do	do	—	—	May 1968	5.1	1,411.06	1.82	86	.92	1,682	
51-26b	Honey Creek Site No. 11	Honey Creek	McKinney, Tex.	1.99	1.92	Feb. 1952	—	1,291.0	1.89	—	—	—	SCS
	do	do	do	—	—	Apr. 1962	10.2	1,201.8	1.76	—	—	—	
	do	do	do	—	—	July 1967	5.3	1,186.7	1.74	51.9	1.48	3,956	
51-27b	Honey Creek Site No. 12	Trib. E. Fork Trinity	do	1.28	1.26	Jan. 1952	—	553.33	1.27	—	—	—	SCS
	do	do	do	—	—	July 1969	17.5	475.52	1.09	40.72	3.53	3,130	
51-28a	Dawson City Lake	Trib. of Richland Creek	Dawson, Tex.	1.16	1.07	Apr. 1937	—	610	1.473	—	—	—	SCS
	do	do	do	—	—	Feb. 1956	18.8	466	1.126	—	7.16	5,130	
	do	do	do	—	—	Sept. 1963	7.5	390.1	.942	--	9.46	6,387	
51-29	Lake Cherokee	Sabine River	Longview, Tex.	170.00	163.77	Oct. 1948	—	49,295	.426	—	—	—	SCS
	do	do	do	—	—	Apr. 1960	11.5	46,705	.412	83.12	1.37	2,480	
51-30	Lavon Reservoir	East Fork Trinity River	Lavon, Tex.	777.0	745.7	— 1945 1/	—	425,400	1.329	—	—	—	CE
	do	do	do	—	—	Oct. 1953 2/	0	434,450	1.363	—	—	—	
	do	do	do	—	—	Oct. 1959	6.0	425,960	1.337	53.1	1.90	2,197	
51-31a	Site No. 11B Elm Fk. Watershed	Trinity River	Gainesville, Tex.	2.00	1.87	Nov. 1958	—	678.2	1.33	—	—	—	SCS
	do	do	do	—	—	July 1963	4.67	670.0	1.31	3/78.3	.96	3/1,637	
	do	do	do	—	—	Oct. 1968	5.25	654.6	1.28	87	1.57	2,402	
51-32	Garza-Little Elm Reservoir (Lewisville Dam).	Elm Fork of Trinity River	Lewisville, Tex.	1,660	1,599	— 1946	—	1,016,200	2.311	—	—	—	CE
	do	do	do	—	—	— 1952 4/	—	1,002,900	2.281	—	—	—	
	do	do	do	—	—	Nov. 1954 5/	—	1,003,400	2.282	—	—	—	
	do	do	do	—	—	Sept. 1960	5.88	989,728	2.251	50.6	1.46	1,609	
51-33	Dam B Reservoir	Neches	Jasper, Tex.	6/7,573	7,546.6	— 1947	—	124,700	.032	—	—	—	CE
	do	do	do	—	—	— 1951	—	7/131,142	.034	—	—	—	
	do	do	do	—	—	Feb. 1960	8.83	131,681	.034	60.9	8/.025	8/33.16	
51-34	Clear Creek Watershed Site No. 21.	Trinity River	Muenster, Tex.	1.54	1.46	May 1962	—	479.4	1.37	—	—	—	SCS
	do	do	do	—	—	Aug. 1967	5.19	468.8	1.34	—	1.40	—	
51-35	Upper Lake Fk. Watershed Site 23	Sabine River	Sulphur Springs, Tex.	9.39	8.80	Aug. 1962	—	3,303.5	.69	—	—	—	SCS
	do	do	do	—	—	Oct. 1967	5.2	3,282.9	.69	—	.45	652	
51-36	Lake Amon G. Carter	Big Sandy Creek	Bowie, Tex.	103	101	May 1956	—	15,805.75	1.07	—	—	—	SCS
	do	do	do	—	—	May 1967	11	15,238.06	1.03	55.19	.51	613	
51-37	Chambers Creek Site 37	Trinity River	Cleburne, Tex.	2.05	1.95	Dec. 1960	—	624.84	1.34	—	—	—	SCS
	do	do	do	—	—	Sept. 1964	3.8	619.37	1.33	—	—	—	
	do	do	do	—	—	May 1969	4.7	617.89	1.33	49	.16	448	
51-38	Chambers Creek Site 101-A	do	Frost, Tex.	2.58	2.34	Apr. 1960	—	1,152.11	1.20	—	—	—	SCS
	do	do	do	—	—	Oct. 1964	4.5	1,137.61	1.18	—	—	—	
	do	do	do	—	—	Sept. 1968	3.9	1,114.89	1.16	54.9	2.19	2,606	
51-39	Clear Fork Watershed Site No. 7.	do	Poolville, Tex.	2.55	2.44	Oct. 1955	—	746.9	1.57	—	—	—	SCS
	do	do	do	—	—	Apr. 1969	13.5	626.29	1.32	47.2	3.66	3,760	
51-40	Denton Creek Watershed Site No. 17.	do	Decatur, Tex.	4.10	3.94	Oct. 1963	—	1,621.33	2.7	—	—	—	SCS
	do	do	do	—	—	Oct. 1968	5	1,557.16	2.6	—	3.26	—	
51-41	East Keechi Creek Site No. 1	Brazos River	Jacksboro, Tex.	6.63	6.30	Jan. 1964	—	1,845.56	1.76	—	—	—	SCS
	do	do	do	—	—	June 1968	4.4	1,827.15	1.74	*72	.66	1,035	
LOWER BRAZOS, LOWER COLORADO, GUADALUPE, SAN ANTONIO, AND NUECES RIVER BASINS													
53-1	Lake Corpus Christi	Nueces River	Mathis, Tex.	16,800	16,751	July 1934	—	54,426	.080	—	—	—	SCS
	do	do	do	—	—	Mar. 1942	7.6	43,801	.064	36.9	.083	63	
	do	do	do	—	—	Mar. 1948	6.0	39,387	.058	35.6	.044	36	
53-2	Medina Lake	Medina River	San Antonio, Tex.	587	578	Mar. 1913	—	274,065	—	—	—	—	SCS
	do	do	do	—	—	Jan. 1937	23.9	267,630	—	61.1	.465	619	
	do	do	do	—	—	May 1948	11.3	265,075	—	75.0	.391	—	

52-3	Buchanan	Colorado River	Burnet, Tex.	9/19,390	19,313	June	1937	—	970,010	*.649	—	—	—	SCS
	do	do	do	—	—	Feb.	1961	3.7	954,859	*.639	—	.21	—	
52-4	Moss Ranch Stock Pond	S. Bull Creek	Llano, Tex.	.07	.07	—	1909	—	4.7	—	—	—	—	SCS
	do	do	do	—	—	Feb.	1961	32	4.5	—	—	.07	—	
52-5	Moss Ranch Stock Pond	W. Bull Creek	do	.20	.19	—	1903	—	9.3	—	—	—	—	SCS
	do	do	do	—	—	Feb.	1961	38	9.1	—	—	.021	—	
52-6	Helms Tank	Trib. of Sandy Creek	do	.15	.14	—	1916	—	12	—	—	—	—	SCS
	do	do	do	—	—	Feb.	1961	25	11.6	—	—	.12	—	
52-7	Baker Lake	Trib. of San Antonio River	Somerset, Tex.	3.17	3.10	Sept.	1950	—	251	.595	—	—	—	SCS
	do	do	do	—	—	Aug.	1955	4.95	228	.540	34.65	1.50	1,132	
52-8a	Calaveras Creek Site No. 6	Calaveras Creek	San Antonio, Tex.	7.01	6.70	Dec.	1956	—	1,697.82	1.65	—	—	—	SCS
	do	do	do	—	—	Mar.	1968	11.25	1,661.12	1.62	66.17	.49	706	
52-9	Escondido No. 11	Trib. of Escondido Creek	Kenedy, Tex.	8.43	7.94	July	1958	—	2,728.0	2.022	—	—	—	SCS
	do	do	do	—	—	July	1960	2.0	2,667.8	1.978	47.08	3.79	3,886	
52-10	Rudder Pond	Trib. of Llano River	Hext, Tex.	1.35	1.34	Aug.	1951	—	11.96	.081	—	—	—	SCS
	do	do	do	—	—	Aug.	1955	4	11.45	.078	—	.09	—	
52-11	Sirianni Lake	Trib. of Hondo Creek	Moore, Tex.	.769	.747	Feb.	1951	—	80.3	1.959	—	—	—	SCS
	do	do	do	—	—	Sept.	1955	4.54	76.5	1.866	45.6	1.12	1,112	
52-12	Smith Pond	Trib. of Benito Creek	Jourdanton, Tex.	.191	.188	Apr.	1953	—	6.41	.361	—	—	—	SCS
	do	do	do	—	—	Aug.	1955	2.4	5.60	.315	27.8	1.80	1,090	
52-13	Thornton Lake	Cibolo Creek	Boerne, Tex.	1.54	1.52	Sept.	1952	—	142.4	.578	—	—	—	SCS
	do	do	do	—	—	Aug.	1955	2.91	135.2	.549	—	1.64	—	
52-14	Iley Lake	Guadalupe River	Gonzales, Tex.	2.34	2.32	Jan.	1949	—	10/48.97	.078	—	—	—	SCS
	do	do	do	—	—	June	1964	15.42	43.75	.070	28.3	.15	92	
52-15	Blackwell Lake	Trib. of Deer Creek Guadalupe River Basin.	Cuero, Tex.	2.76	2.75	Sept.	1949	—	10/45.12	.061	—	—	—	SCS
	do	do	do	—	—	June	1964	14.75	41.51	.056	49.6	.09	97	
52-16a	Site No. 1 Escondido Creek	San Antonio River	Kenedy, Tex.	3.01	2.83	Sept.	1954	—	924.7	2.30	—	—	—	SCS
	do	do	do	—	—	June	1964	9.8	906.2	2.26	—	.67	3/763	
	do	do	do	—	—	July	1969	5.1	890.1	2.22	—	1.12	1,710	
52-17	Cummings Creek Watershed Site No. 6.	Trib. Cummings Creek	Giddings, Tex.	2.99	2.76	Aug.	1958	—	858.3	.77	—	—	—	SCS
	do	do	do	—	—	Sept.	1963	5.1	855.2	.77	—	.22	—	
	do	do	do	—	—	Aug.	1969	5.9	854.1	.76	—	.07	—	
	BRAZOS RIVER BASIN (SOUTH BEND TO WASHINGTON) MIDDLE, AND COLORADO RIVER BASINS													
53-1	Lake Scarborough	Trib. of Jim Ned Creek	Coleman, Tex.	10.8	10.6	May	1923	—	2,153	—	—	—	—	SCS
	do	do	do	—	—	May	1960	17	2,007	—	51.41	.81	907	
53-2	Santa Anna Lake	Mud Creek	Santa Anna, Tex.	1.17	1.05	May	1923	—	766	—	—	—	—	SCS
	do	do	do	—	—	Apr.	1960	17	745	—	45.2	1.18	1,162	
53-3	Lake Kanes	Mercer Creek	Comanche, Tex.	13.76	13.57	May	1926	—	1,313	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1946	20.3	1,221	—	—	.334	—	
53-4	Lake Merritt	Browns Creek	Goldthwaite, Tex.	11.65	11.50	May	1917	—	962	—	—	—	—	SCS
	do	do	do	—	—	May	1960	23	855	—	43	.40	375	
53-5	Hubbard City Lake No. 3	Trib. of E. Cottonwood Cr.	Hubbard, Tex.	.16	.14	—	1913	—	11/104.5	—	—	—	—	SCS
	do	do	do	—	—	May	1949	36	90.2	—	—	2.86	—	
53-6	Hubbard City Lake No. 4	E. Cottonwood Creek	do	1.40	1.35	—	1917	—	11/318.2	—	—	—	—	SCS
	do	do	do	—	—	May	1949	32	255.4	—	—	1.45	—	
53-7a	Rogers Lake	Trib. of Little River	Rogers, Tex.	.55	.51	Fall	1922	—	164	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1934	12	126.5	—	—	6.12	—	
53-8	Hubbard City Lake No. 1	Trib. E. Cottonwood Creek	Hubbard, Tex.	.5	.5	—	1898	—	12/110	—	—	—	—	SCS
	do	do	do	—	—	May	1949	51	84	—	—	1.0	—	
53-9	Hubbard City Lake No. 2	do	do	.03	.03	—	1912	—	12/36.3	—	—	—	—	SCS
	do	do	do	—	—	May	1949	37	33.0	—	—	3.0	—	
53-10	Hubbard City Lake No. 5	East Cottonwood Creek	do	.11	.10	—	1925	—	44.8	—	—	—	—	SCS
	do	do	do	—	—	May	1949	24	37.8	—	—	3.0	—	
53-11	Lake Leon	Redbank Creek	Leon, Tex.	13.0	12.8	—	1921	—	530	—	—	—	—	SCS
	do	do	do	—	—	Mar.	1961	30	354	—	—	.46	—	
53-12	Lometa	Salt & Emory Creeks	Lometa, Tex.	4.74	4.60	—	1912	—	759	—	—	—	—	SCS
	do	do	do	—	—	Feb.	1941	29	732	—	—	.20	—	
53-13	Meridian Lake	Bee Creek	Meridian, Tex.	3.30	3.20	—	1934	—	723	—	—	—	—	SCS
	do	do	do	—	—	Apr.	1948	14	692	—	—	.69	—	
53-14	Miller Lake	Trib. of Horse Creek	San Saba, Tex.	.41	.38	Sept.	1913	—	83	—	—	—	—	SCS
	do	do	do	—	—	Mar.	1941	27.5	75	—	—	.71	—	
53-15	Odell Lake	Unnamed trib.	Hubbard, Tex.	.56	.55	July	1925	—	75.1	—	—	—	—	SCS
	do	do	do	—	—	July	1949	24	52.6	—	—	1.71	—	
53-16	Possum Kingdom Lake	Brazos River	Graford, Tex.	13/14,098	12,955	May	1941	—	729,985	—	—	—	—	SCS
	do	do	do	—	—	Feb.	1969	7.75	672,420	—	—	.57	—	
53-17	Rock Crusher	Bachelor Creek	Coleman, Tex.	16.5	16.48	—	1910	—	153	—	—	—	—	SCS
	do	do	do	—	—	Feb.	1941	31	80	—	—	*.14	—	
53-18	Old Santa Anna City Lake	Mukewater Creek	Santa Anna, Tex.	.90	.87	Jan.	1910	—	153	—	—	—	—	SCS
	do	do	do	—	—	June	1940	30.5	118	—	—	1.31	—	

1/ Original capacity from map by stereo photogrammetric methods.

2/ "Original" or 1965 capacity adjusted in 1961 by range line controls established in 1953 give "Oct. 53" area and capacity.

3/ Sediment pool only.

4/ Adjusted for 1952 survey of Lake Dallas.

5/ Adjusted in Apr. 1963 for detailed range survey in 1952-53-54.

6/ Determined by USGS Nov. 1961.

7/ 1951 adjusted data.

8/ Deposits above Highway 190 bridge only. Not corrected for degradation between highway and dam.

9/ Drainage area is 31,250 sq. mi., of which 11,900 sq. mi. are noncontributing.

10/ Original capacity determined by spadding on 1964 survey.

11/ Dam was raised in 1925 and 1949. Capacities based on 1949 level.

12/ Dam was raised in 1925. Capacity based on 1925 level.

13/ Includes 1,111 sq. mi. of partially contributing drainage above lakes in watershed; excludes 8,900 sq. mi. of noncontributing drainage at head of watershed.

* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
BRAZOS RIVER BASIN (SOUTH BEND TO WASHINGTON) MIDDLE, AND COLORADO RIVER BASINS (Continued)													
53-19	Old Coleman City Lake	Home Creek	Coleman, Tex.	0.73	0.69	Nov. 1926	—	289	—	—	—	—	SCS
	do	do	do	—	—	May 1960	33.6	273	—	—	0.67	—	
53-20	Hamilton City Lake	Ten Mile Creek	Hamilton, Tex.	12.0	11.9	June 1923	—	610	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1961	17.75	560	—	—	.24	—	
53-21	Lake Leon	Leon River	Eastland, Tex.	225	224.7	June 1920	—	3/1,437	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1941	20.75	1,275	—	—	.08	—	
53-22	Lake Mineral Wells	Rock Creek	Mineral Wells, Tex.	74.4	73.4	— 1922	—	10,741	—	—	—	—	SCS
	do	do	do	—	—	Dec. 1941	19.5	9,032	—	—	1.19	—	
53-23	Buffalo Tank (Knox Tank)	Trib. of Pecan Bayou	Coleman, Tex.	1.73	1.71	— 1920	—	33	—	—	—	—	SCS
	do	do	do	—	—	Feb. 1961	41	29	—	—	.058	—	
53-24	R. C. Hollingsworth Stock Pond.	do	do	2.60	2.58	June 1937	—	20	—	—	—	—	SCS
	do	do	do	—	—	Feb. 1961	3.7	14	—	—	.46	—	
53-25	J. S. Wall Stock Pond	Trib. of Brady Creek	Brady, Tex.	.35	.35	— 1927	—	13	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1941	14	12.2	—	—	.16	—	
53-26	White Tank	Trib. of Pecan Bayou	Brownwood, Tex.	.80	.80	May 1936	—	4.5	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1941	4.8	3.6	—	—	.24	—	
53-27	Zimmerlee Stock Pond (North)	Trib. of Jim Ned Creek	Lawn, Tex.	.13	.13	— 1916	—	1.12	—	—	—	—	SCS
	do	do	do	—	—	Feb. 1941	25	.80	—	—	.10	—	
53-28	Smith Lake	Redbank Creek	do	1.04	1.02	Aug. 1926	—	102	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1941	14.6	93	—	—	.63	—	
53-29	Philpott Lake	Paint Creek	Pioneer, Tex.	9.04	9.00	Apr. 1925	—	183	—	—	—	--	SCS
	do	do	do	—	—	Feb. 1941	15.9	173	—	—	.07	—	
53-30b	Lake Brownwood	Pecan Bayou (Colo. River)	Brownwood, Tex.	1,544	1,533	July 1932	—	149,925	1.011	—	—	—	SCS
	do	do	do	—	—	Feb. 1940	7.6	145,720	.983	41.9	.361	329	
	do	do	do	—	—	Sept. 1959	19.6	135,963	.917	42.2	.325	299	
53-31a	Lake Waco (Old)	Bosque	Waco, Tex.	2/1,649	2/1,645	Apr. 1930	—	39,378	—	—	—	—	CE
	do	do	do	—	—	Feb. 1935	4.9	33,717	—	*58.5	.695	886	
	do	do	do	—	—	Feb. 1936	1.0	31,580	—	*58.5	1.28	1,631	
	do	do	do	—	—	Dec. 1947	11.8	22,026	—	*58.5	.487	621	
	do	do	do	—	—	Dec. 1964	17.0	15,427	—	—	.236	—	
53-32	Lake Eddleman	Flint Creek	Graham, Tex.	42	41.4	Feb. 1929	—	6,583	1.144	—	—	—	SCS
	do	do	do	—	—	May 1954	25.25	5,917	1.028	49.2	.64	687	
53-33b	Site No. 3 Cow Bayou	Brazos	Moody, Tex.	1.40	1.32	Nov. 1955	—	458.5	1.23	—	—	—	SCS
	do	do	do	—	—	Aug. 1960	4.75	393.3	1.05	—	10.40	—	
	do	do	do	—	—	Aug. 1965	5.04	375.4	1.00	—	2.69	—	
	do	do	do	—	—	Apr. 1970	4.69	366.7	.98	—	1.41	1,572	
53-34a	Cow Bayou No. 4	Foster Creek	Waco, Tex.	5.25	5.20	July 1956	—	1,915	1.26	—	—	—	SCS
	do	do	do	—	—	Sept. 1969	13.2	1,833.53	1.19	—	1.19	1,419	
53-35	Deep Creek No. 3	Trib. of Deep Creek	Brady, Tex.	3.42	3.19	Nov. 1953	—	925.18	2.821	—	—	—	SCS
	do	do	do	—	—	Aug. 1960	6.8	881.80	2.688	76.57	2.00	3,335	
53-36a	Green Creek No. 1	Green Creek	Dublin, Tex.	3.57	3.38	Apr. 1955	—	1,095	1.98	—	—	—	SCS
	do	do	do	—	—	Jan. 1957	1.8	1,080	1.96	—	—	—	
	do	do	do	—	—	Apr. 1962	5.2	1,065	1.93	—	—	—	
	do	do	do	—	—	June 1967	5.2	1,063	1.93	73.6	.11	3/80	
53-37	Silver Lake	N. E. Trib. of Leon River	do	.37	.334	— 1910	—	153.8	2.435	—	—	—	SCS
	do	do	do	—	—	July 1960	50	145.0	2.296	—	.527	—	
53-38	Main Res. Deep Creek Site 5	Colorado	Brady, Tex.	2.92	2.72	Aug. 1953	—	1,326.7	4.755	—	—	—	SCS
	do	do	do	—	—	July 1961	7.9	1,295.7	4.644	—	1.44	—	
53-39	Upper Res. Deep Creek Site 5	do	do	2.19	2.18	Aug. 1953	—	15.2	.072	—	—	—	SCS
	do	do	do	—	—	July 1961	7.9	11.2	.053	49.9	.23	290	
53-40a	Mukewater Creek Site No. 9	Mid. Colorado River	Bangs, Tex.	4.75	4.55	Jan. 1961	—	732.94	1.38	—	—	—	SCS
	do	do	do	—	—	June 1965	4.42	708.42	1.33	—	1.22	—	
	do	do	do	—	—	July 1970	5.1	703.41	1.32	—	.22	—	
53-41a	Sulphur Creek Site 3	Lampasas River	Lampasas, Tex.	10.81	10.58	Dec. 1959	—	3,229.2	1.6	—	—	—	SCS
	do	do	do	—	—	Nov. 1962	2.8	3,226.7	1.6	—	—	—	
	do	do	do	—	—	Aug. 1968	5.8	3,223.3	1.6	55	3/.02	3/26	
53-42	Whitney Reservoir	Brazos River	Whitney, Tex.	4/17,656	5/3,480	Dec. 1951	—	6/2,013,600	—	—	—	—	CE
	do	do	do	—	—	Apr. 1959	7.4	1,999,500	—	62.6	.55	749.68	
53-43	Site No. 6 Deep Creek	Dry Prong	Brady, Tex.	4.26	4.02	Dec. 1951	—	1,409.41	2.70	—	—	—	SCS
	do	do	do	—	—	Sept. 1966	14.8	1,367.81	2.62	—	.71	—	
53-44	Site No. 10-A Mukewater Creek	Mukewater Creek	Bangs, Tex.	15.26	14.59	Mar. 1965	—	3,164.89	1.85	—	—	—	SCS
	do	do	do	—	—	Aug. 1966	1.4	3,157.79	1.85	—	.34	—	
53-45	Lake Daniel	Gonzales Creek	Breckenridge, Tex.	115	113	June 1949	—	10,731	.81	—	—	—	SCS
	do	do	do	—	—	Nov. 1970	21.4	9,515	.72	48	.49	512	
53-46	Site 9 Lower San Saba River	Colorado River	San Saba, Tex.	3.03	2.88	Jan. 1960	—	665.79	1.60	—	—	—	SCS
	do	do	do	—	—	Sept. 1967	7.67	635.49	1.57	—	.46	720	

UPPER BRAZOS AND UPPER COLORADO RIVER BASINS

54-1	Lake Abilene	Elm Creek	Abilene, Tex.	98.5	97.5	Aug. 1921	—	10,325	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1948	27	9,786	—	*60	.21	274	
54-2b	Lake Nasworthy	South Concho	San Angelo, Tex.	3,294	3,119	Oct. 1930	—	11,485	*.046	—	—	—	SCS
	do	do	do	—	—	Dec. 1938	8.2	10,294	*.041	67.5	.046	68	
	do	do	do	—	—	May 1953	14.5	7/12,628	*.051	67.5	.023	34	
54-3	Lake Sweetwater	Bitter Creek	Sweetwater, Tex.	110	108.8	Feb. 1930	—	13,810	—	—	—	—	SCS
	do	do	do	—	—	Dec. 1941	11.8	13,287	—	—	.41	—	
54-4	Lake Kirby	Cedar Creek	Abilene, Tex.	44	42.8	Sept. 1928	—	8,133	—	—	—	—	SCS
	do	do	do	—	—	Nov. 1941	13.2	7,618	—	—	.51	—	
54-5	Lake Throckmorton	South Elm River	Throckmorton, Tex.	11.54	11.32	Sept. 1918	—	8/1,967	1.534	—	—	—	SCS
	do	do	do	—	—	June 1954	35.7	1,829	1.441	31.55	.29	199	
54-6a	Lake Fort Phantom Hill	Elm Fork, Brazos River	Abilene, Tex.	478	268	Oct. 1938 9/	—	76,966	—	—	—	—	SCS
	do	do	do	—	—	Aug. 1953	14.8	74,312	—	35.99	.67	525	
54-7	Mountain Creek	Mountain Creek	Robert Lee, Tex.	32.00	31.88	May 1950	—	945.3	.369	—	—	—	SCS
	do	do	do	—	—	Apr. 1961	10.9	804.9	.314	53.06	.40	462.17	
54-8	KJK Ranch Pond	Trib. of S. Concho River	San Angelo, Tex.	1.60	1.53	May 1937	—	276	2.240	—	—	—	SCS
	do	do	do	—	—	Aug. 1955	18.2	250	2.029	63.6	.93	1,288	
54-9	Lake Stamford	Paint Creek	Stanford, Tex.	360	352.34	June 1953	—	57,632.35	1.71	—	—	—	SCS
	do	do	do	—	—	May 1966	12.9	53,927.57	1.60	42.56	.82	760	
54-10	Site 18 Valley Cr. Watershed	Trib. Valley Creek	Winters, Tex.	4.21	4.11	Aug. 1964	—	719.89	2.38	—	—	—	SCS
	do	do	do	—	—	June 1969	4.8	717.05	2.36	69	.14	210	
54-11	Lake Ballinger	Valley Creek	Ballinger, Tex.	210	209	Nov. 1947	—	3,872.09	.28	—	—	—	SCS
	do	do	do	—	—	Aug. 1970	22.8	3,070.42	.22	37	.17	137	
54-12	Lake Winters	Elm Creek	Winters, Tex.	63.62	63.14	Oct. 1945	—	2,518.2	.59	—	—	—	SCS
	do	do	do	—	—	July 1970	24.75	1,886.2	.44	39	.40	340	

RIO GRANDE BASIN (BELOW EAGLE PASS)

55-1	Site 6 Olmitos & Garcias Creeks	El Gato Creek	Rio Grande City, Tex.	13.19	12.94	June 1962	—	2,258.11	21.5	—	—	—	SCS
	do	do	do	—	—	Sept. 1966	4.25	2,252.03	21.4	6/75	.11	179.69	

RIO GRANDE BASIN (FORT QUITMAN TO EAGLE PASS) AND LOWER PECOS RIVER BASIN

56-1a	Cottonwood Detention	San Felipe Arroyo River	Fabens, Tex.	2.34	2.19	July 1937	—	500	16.129	—	—	—	SCS
	do	do	do	—	—	July 1954	17	378	12.154	—	3.28	—	
56-2	Rattlesnake Detention	do	do	5.15	10/2.81	July 1937	—	224.4	—	—	—	—	SCS
	do	do	do	—	—	July 1954	17	175.7	—	—	1.02	—	
56-3	Roberts Detention	do	do	2.34	2.31	July 1937	—	145.3	4.687	—	—	—	SCS
	do	do	do	—	—	July 1954	17	104.8	3.381	—	1.03	—	
56-4a	Fresno Pond	Trib. of Devils River	Sonora, Tex.	.435	.430	Dec. 1944	—	7.31	.222	—	—	—	SCS
	do	do	do	—	—	Aug. 1955	10.75	6.97	.211	69.6	.074	112	
56-5	Lowrey Draw, Site No. 9	Lowrey Draw	do	2.77	2.64	June 1961	—	3/31.1	.49	—	—	—	ARS
	do	do	do	—	—	Feb. 1969	7.65	30.84	.48	64	.01	17.5	
56-6	Lowrey Draw, Site No. 10	Trib. Lowrey Draw	do	8.43	8.20	June 1961	—	3/95	.68	—	—	—	ARS
	do	do	do	—	—	Mar. 1969	7.83	94.38	.68	64	.01	13.5	
56-7	Lowrey Draw, Site No. 12	do	do	4.38	4.19	June 1961	—	3/85.46	.68	—	—	—	ARS
	do	do	do	—	—	Feb. 1969	7.65	84.03	.67	64	.04	62.2	
56-8	Lowrey Draw, Site No. 13	do	do	1.07	1.02	June 1961	—	3/56.26	3.28	—	—	—	ARS
	do	do	do	—	—	Mar. 1969	7.74	56.10	3.26	64	.02	28.2	
56-9	Site No. 1 Diablo Arroyo	Rio Grande	McNary, Tex.	29.89	29.51	Apr. 1960	—	4,489.7	11.3	—	—	—	SCS
	do	do	do	—	—	Apr. 1965	5	4,343.1	10.9	—	—	—	
	do	do	do	—	—	Apr. 1970	5	4,235.1	10.6	—	.86	1,129.5	

1/ Dam was raised 1 ft. in 1928. Capacity at 1928 spillway level.
2/ Latest drainage area by USGS in 1964.
3/ Sediment pool only.
4/ Does not include 8,900 sq. mi. which is considered to be noncontributing to the total 26,[illegible] sq. mi. drainage area.
5/ For the 3,598 sq. mi. area between Possum Kingdom and Whitney Dams, excluding 77.8 sq. mi. which is reservoir surface area at elevation 571.
6/ Original storage capacity 2,017,500 ac.-ft. Adjusted in 1963 for detailed range survey in 1951.
7/ Spillway raised 2.3 ft. in Sept. 1948.
8/ Dam was raised 13 ft. in 1940; capacity at 1940 spillway elevation.
9/ Date storage began has not been verified.
10/ Includes Hueco Watershed but does not include Roberts Watershed.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
						RIO GRANDE BASIN (ESPANOLA TO FORT QUITMAN)							
57-1b	Elephant Butte	Rio Grande River	Elephant Butte, N. Mex.	25,923	25,866	Jan. 1915	—	2,634,800	—	—	—	—	BR
	do	do	do	—	—	Dec. 1916	1.9	2,584,865	—	—	1.02	—	
	do	do	do	—	—	Aug. 1920	3.7	2,498,850	—	—	.899	—	
	do	do	do	—	—	Aug. 1925	5.0	2,389,380	—	—	.866	—	
	do	do	do	—	—	Apr. 1935	9.7	2,270,300	—	460	.475	621	
	do	do	do	—	—	Oct. 1940	5.5	2,219,000	—	—	.361	—	
	do	do	do	—	—	Apr. 1947	6.5	2,197,600	—	*65.9	.127	182	
	do	do	do	—	—	Feb. 1957	9.75	3/2,206,780	2.20	60.0	—	—	
	do	do	do	—	—	Apr. 1969	12.2	2,137,219	2.31	62	.22	357	
57-2	Caballo	do	Truth or Consequences, N. Mex.	2/30,700	3/1,237	Jan. 1938	—	345,872	—	—	—	—	BR
	do	do	do	—	—	Nov. 1957	19.9	343,990	—	—	.0761	—	
57-3a	Santa Cruz	Santa Cruz River	Espanola, N. Mex.	93.1	93.1	Feb. 1929	—	4,464	.190	—	—	—	SCS
	do	do	do	—	—	Aug. 1956	27.4	3,758	.160	80	.27	470	
57-4	Cornfield Wash (1)	Trib. of Rio Puerco River	Cuba, N. Mex.	.29	.28	Apr. 1951	—	24.0	1.446	—	—	—	GS
	do	do	do	—	—	Oct. 1960	4/10	10.3	.620	—	4.69	—	
57-5	Cornfield Wash (2)	do	do	.87	.85	Apr. 1951	—	54.1	3.127	—	—	—	GS
	do	do	do	—	—	Oct. 1960	4/10	45.1	2.607	—	1.06	—	
57-6	Cornfield Wash (3)	do	do	.25	.25	Apr. 1951	—	5.9	.721	—	—	—	GS
	do	do	do	—	—	Oct. 1960	4/10	2.6	.313	—	1.32	—	
57-7	Cornfield Wash (4)	do	do	1.18	1.17	Apr. 1951	—	22.1	1.277	—	—	—	GS
	do	do	do	—	—	Oct. 1960	4/10	17.4	1.006	—	.40	—	
57-8	Cornfield Wash (5)	do	do	1.04	1.04	Apr. 1951	—	9.2	.476	—	—	—	GS
	do	do	do	—	—	Oct. 1960	4/10	3.0	.155	—	.60	—	
57-9	Cornfield Wash (9)	do	do	.09	.09	Apr. 1951	—	4.6	.948	—	—	—	GS
	do	do	do	—	—	Oct. 1960	4/10	3.1	.639	—	1.67	—	
57-10	Cornfield Wash (13)	do	do	.33	.33	Apr. 1951	—	7.4	.361	—	—	—	GS
	do	do	do	—	—	Oct. 1960	4/10	.3	.023	—	2.15	—	
57-11	Cornfield Wash (15)	do	do	1.04	1.03	Dec. 1953	—	17.9	6.630	—	—	—	GS
	do	do	do	—	—	Oct. 1960	4/7	17.8	6.593	—	.01	—	
57-12	San Luis (1)	do	do	1.06	1.05	Aug. 1952	—	16.3	.676	—	—	—	GS
	do	do	do	—	—	Oct. 1960	4/8	12.0	.498	—	.51	—	
57-13	San Luis (3)	do	do	.67	.66	Aug. 1952	—	22.6	.958	—	—	—	GS
	do	do	do	—	—	Oct. 1960	4/8	19.6	.831	—	.50	—	
57-14	Zia Reservoir	Trib. of Jemez River	San Ysidro, N. Mex.	2.4	2.4	June 1954	—	59.9	2.155	—	—	—	GS
	do	do	do	—	—	Oct. 1960	4/7	19.0	.683	—	2.43	—	
57-15a	Jemez Canyon Reservoir	Jemez River	Bernalillo, N. Mex.	1,034	1,034	Oct. 1953	—	117,213	2.844	—	—	—	CE
	do	do	do	—	—	Jan. 1958	5/4.25	115,822	2.810	*75.7	.317	523	
	do	do	do	—	—	Aug. 1959	5/1.67	113,874	2.763	*75.7	1.128	1,860	
	do	do	do	—	—	Dec. 1965	5/6.33	112,809	2.737	*75.7	.162	267	
57-16	San Luis #2	Rio Puerco River	Cuba, N. Mex.	.74	.74	July 1952	—	*3.0	.120	—	—	—	FS
	do	do	do	—	—	July 1962	—	—	—	—	.62	—	
57-17	Caballo Arroyos Site 1 (Underwood).	Underwood Arroyo	Derry, N. Mex.	.87	.87	Nov. 1959	—	73.21	15.9	—	—	—	SCS
	do	do	do	—	—	Aug. 1967	7.8	68.79	14.9	111	.44	1,547	
57-18	Site #6 (Alamo Arroyo)	Alamo Arroyo	Chimayo, N. Mex.	3.15	3.12	Apr. 1962	—	232	6.9	—	—	—	SCS
	do	do	do	—	—	Dec. 1968	6.7	160	4.76	*100	3.30	7,187	
57-19	Site #3 (Ramones Arroyo)	Ramones Arroyo	do	1.17	1.16	Sept. 1962	—	111	8.9	—	—	—	SCS
	do	do	do	—	—	Jan. 1969	6.3	69	5.5	*110	6.03	14,446	
	do	do	do	—	—	Oct. 1969	.7	37.5	3.0	*110	27.16	102,597	
57-20	North Salem Site #2	North Salem Arroyo	Salem, N. Mex.	3.8	3.8	Apr. 1957	—	204.71	*10.1	—	—	—	SCS
	do	do	do	—	—	Aug. 1967	10.25	190.82	*9.4	98.5	.35	750.9	
57-21	Rodey (Hatch Valley Arroyo #5)	Rodey Arroyo	Hatch, N. Mex.	2.1	2.1	July 1958	—	170.91	15.4	—	—	—	SCS
	do	do	do	—	—	Aug. 1967	9.1	161.25	12.7	85	1.55	2,889	
						UPPER PECOS RIVER BASIN							
58-1	Bonito	Bonito Creek, Kraut Gulch	Capitan, N. Mex.	40.0	39.9	June 1931	—	1,180	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1940	9	1,150	—	—	.083	—	
58-2a	Alamogordo	Pecos River	Ft. Sumner, N. Mex.	4,393	3,749	— 1936	—	156,750	.946	—	—	—	BR
	do	do	do	—	—	Sept. 1940	3.25	146,071	.882	*73.5	.876	1,410	
	do	do	do	—	—	Oct. 1943	3.08	130,280	.786	—	—	—	
	do	do	do	—	—	Apr. 1944	0.50	132,171	.797	*73.5	1.03	1,649	
	do	do	do	—	—	Sept. 1964	20.42	110,655	.668	*73.5	.282	450	
58-3a	Avalon (Lake Avalon)	Pecos River	Carlsbad, N. Mex.	16,030	1,080	Apr. 1907	—	7,600	—	—	—	—	BR
	do	do	do	—	—	Apr. 1941	34.00	6,633	—	—	6/.0263	—	
	do	do	do	—	—	July 1956	15.25	5,647	—	76.7	.0399	—	

58-4a	McMillan (Lake McMillan)	do	do	14,950	14,950	Jan.	1894	—	91,000	.344	—	—	—	BR
	do	do	do	—	—	June	1904	10.42	73,000	.276	—	.116	—	
	do	do	do	—	—	Nov.	1910	6.42	61,500	.232	—	.120	—	
	do	do	do	—	—	May	1915	4.90	45,500	.172	—	.238	—	
	do	do	do	—	—	June	1925	10.08	42,000	.159	—	.023	—	
	do	do	do	—	—	Dec.	1932	7.50	40,500	.153	—	.013	—	
	do	do	do	—	—	Jan.	1940	7.08	38,655	.146	*70	.027	—	
	do	do	do	—	—	July	1956	16.50	7/39,400	.149	8/62.4	.004	—	
58-5	Upper-Hondo Site #1	Salado Creek	Capitan, N. Mex.	122	93.92	May	1959	—	4,972.14	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1959	.42	4,946.54	—	*75	.65	1,060	
	do	do	do	—	—	May	1961	1.58	4,896.60	—	75	.34	555	
	do	do	do	—	—	Mar.	1962	.83	4,818.00	—	75	1.01	1,649	
	do	do	do	—	—	Feb.	1963	.92	4,796.41	—	75	.25	408	
	do	do	do	—	—	Jan.	1965	1.92	4,749.54	—	75	.26	424	
58-6	Bancroft #7	do	do	12	12	Feb.	1955	—	4.77	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1965	10.67	3.52	—	*75	.01	16	
58-7	Bancroft #9	do	do	.57	.57	Feb.	1955	—	7.20	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1965	10.67	1.59	—	*75	.92	1,500	
58-8	Bancroft #11	do	do	.69	.69	Feb.	1955	—	4.09	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1965	10.67	1.59	—	*75	.34	554	
58-9	Pearson #6	do	do	.19	.19	Mar.	1955	—	2.21	—	—	—	—	SCS
	do	do	do	—	—	May	1957	2.17	1.60	—	*75	1.46	2,380	
	do	do	do	—	—	Apr.	1959	1.91	1.52	—	*75	.24	391	
	do	do	do	—	—	Oct.	1965	6.50	.49	—	*75	.83	1,352	
58-10	Pearson #8	do	do	.24	.24	Mar.	1955	—	5.45	—	—	—	—	SCS
	do	do	do	—	—	Dec.	1956	1.75	4.30	—	*75	2.75	4,555	
	do	do	do	—	—	Jan.	1958	1.08	3.62	—	*75	2.63	4,280	
	do	do	do	—	—	Jan.	1959	1.00	3.02	—	*75	2.49	4,130	
	do	do	do	—	—	Oct.	1965	6.75	.55	—	*75	1.53	2,519	
58-11	Pearson #9	do	do	1.56	1.56	Nov.	1954	—	23.78	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1965	10.92	16.05	—	*75	.45	734	
58-12	Pearson #10	do	do	.71	.71	Mar.	1955	—	20.19	—	—	—	—	SCS
	do	do	do	—	—	May	1957	2.17	18.04	—	*75	1.39	2,262	
	do	do	do	—	—	Jan.	1958	.67	17.99	—	*75	.14	228	
	do	do	do	—	—	Oct.	1958	.75	17.58	—	*75	.76	1,241	
	do	do	do	—	—	Oct.	1965	7.00	16.05	—	*75	.31	505	
58-13	Pearson #18	do	do	1.80	1.80	Mar.	1955	—	15.60	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1965	10.67	2.20	—	*75	.70	1,142	
58-14	Pearson #20	do	do	9/.16	9/.16	Mar.	1955	—	6.55	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1965	10.58	5.77	—	*75	.46	752	

COLORADO RIVER BASIN (BELOW HOOVER DAM)
Williams and Lower Gila River Basins

99-

GILA RIVER BASIN

60-1	Lake Pleasant	Agua Fria River	Phoenix, Ariz.	1,450	1,444	Apr.	1928	—	184,500	—	—	—	—	SCS
	do	do	do	—	—	Feb.	1941	12.9	176,456	—	—	.432	—	
60-2a	San Carlos (Coolidge Dam)	Gila River	Globe, Ariz.	12,900	*11,900	Nov.	1928	—	1,266,837	6.054	—	—	—	GS
	do	do	do	—	—	Feb.	1935	6.3	1,232,725	5.891	—	.455	—	
	do	do	do	—	—	Jan.	1937	1.9	1,230,695	5.882	—	.090	—	
	do	do	do	—	—	Jan.	1947	10.0	1,209,343	5.780	—	.179	—	
	do	do	do	—	—	—	1966	19.6	1,170,118	5.592	—	.168	—	
60-3	Stock Tank No. 16 (Bryce Dam)	Butler Wash	Eden, Ariz.	.69	.69	Mar.	1936	—	11.43	—	—	—	—	SCS
	do	do	do	—	—	—	1941	5.2	5.13	—	—	1.75	—	
60-4	Roosevelt-Salt River Project	Salt River & Tonto Creek	Globe, Ariz.	5,760	5,760	May	1909	—	1,522,200	1.886	—	—	—	BR
	do	do	do	—	—	Dec.	1914	5.7	1,495,460	1.853	*70	.819	1,268	
	do	do	do	—	—	Oct.	1916	1.8	1,460,150	1.809	*70	3.350	5,107	
	do	do	do	—	—	Sept.	1925	8.9	1,425,813	1.767	*70	.670	1,021	
	do	do	do	—	—	Jan.	1935	9.3	1,418,013	1.757	*70	.145	221	
	do	do	do	—	—	Jan.	1939	4.0	1,398,430	1.733	*70	.850	1,296	
	do	do	do	—	—	Jan.	1946	7.0	1,381,580	1.712	*70	.418	637	

1/ Total storage shows a gain of 9,180 ac.-ft. since 1947 survey attributable primarily to compaction.
2/ Includes 2,940 sq. mi. in closed basin in San Luis Valley, Colo.
3/ Drainage area between gaging station below Elephant Butte Dam and gaging station below Caballo Dam less original water surface area of Caballo Reservoir at elevation 4,225.3.
4/ Runoff seasons.
5/ Time periods adjusted.
6/ Compacted sediment for 1,080 sq. mi.; values for 16,030 sq. mi. are given in appendix summaries.
7/ Increase in capacity due mainly to compaction.
8/ Only surface samples (1.0-3.1 ft.) in approximately 1/3 of reservoir area below spillway crest.
9/ Drainage area is 100 ac. plus pipe flow from Pearson #11, which has a drainage area of 10 ac., plus pipe and emergency spillway flow from Pearson #18, which has a drainage area of 1.80 sq. mi.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES) TOTAL	DRAINAGE AREA (SQUARE MILES) NET	DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN AC.-FT.	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN TONS	AGENCY SUPPLYING DATA
			GILA RIVER BASIN (Continued)										
60-5	Allen	Trib. of Gila River	Thatcher, Ariz.	1/4.17	4.13	— 1936	—	279	2.847	—	—	—	SCS
	do	do	do	—	—	June 1958	22	253	2.582	—	0.27	—	
60-6	"P" Mt. Tank No. 1	Whitewater Draw	Douglas, Ariz.	.21	.21	— 1917	—	2.96	.516	—	—	—	SCS
	do	do	do	—	—	Nov. 1958	41	2.56	.446	*75	.05	81.7	
60-7	"P" Mt. No. 2	do	do	10.92	10.92	— 1938	—	2.55	.009	—	—	—	SCS
	do	do	do	—	—	Nov. 1958	20	.70	.002	*75	.008	13.1	
60-8	"P" Mt. No. 3	do	do	2.48	2.48	Sept. 1947	—	14.7	.218	—	—	—	SCS
	do	do	do	—	—	Nov. 1958	2/7	.0	0	*85	.85	1,573	
60-9	"P" Mt. No. 4	do	do	.36	.36	Mar. 1942	—	.63	—	—	—	—	SCS
	do	do	do	—	—	Nov. 1958	2/12	.0	—	*90	.10	196	
60-10	"P" Mt. No. 5	do	do	12.3	12.3	— 1943	—	3.0	.009	—	—	—	SCS
	do	do	do	—	—	Nov. 1958	3/5	2.7	.008	*75	.006	9.8	
60-11	"P" Mt. Pond No. 6	do	do	5.68	5.68	— 1941	—	.8	.005	—	—	—	SCS
	do	do	do	—	—	Nov. 1958	4/6	.6	.004	*75	.006	9.8	
60-12	Dos Cabezas No. 4 SW	Trib. of Willcox Dry Lake	Dos Cabezas, Ariz.	.6	.6	— 1949	—	6.0	.312	—	—	—	SCS
	do	do	do	—	—	Nov. 1960	11	4.1	.214	*90	.29	559	
60-13a	Dos Cabezas No. 15	do	do	.38	.38	— 1940	—	4.7	.385	—	—	—	SCS
	do	do	do	—	—	Oct. 1960	20	2.8	.230	*90	.26	510	
60-14	Halfmoon	Trib. of Gila River	Florence, Ariz.	.2	.2	— 1941	—	6.8	1.889	—	—	—	SCS
	do	do	do	—	—	June 1960	19	3.9	1.083	*85	.75	1,388	
60-15	Homestead	do	do	.9	.9	— 1954	—	6.00	.366	—	—	—	SCS
	do	do	do	—	—	June 1960	6	1.83	.112	*90	.78	1,530	
60-16	Lower Posto Wash	Posto Wash	Safford, Ariz.	4.80	5/.63	— 1936	—	14.8	—	—	—	—	SCS
	do	do	do	—	—	June 1958	22	13.0	—	—	.13	—	
60-17	Magma No. 1	Magma Wash	Florence, Ariz.	1.78	1.78	— 1948	—	5.7	.176	—	—	—	SCS
	do	do	do	—	—	May 1959	11	2.7	.084	*80	.15	261	
60-18	Magma No. 2	do	do	.38	.38	Feb. 1954	—	1.6	.232	—	—	—	SCS
	do	do	do	—	—	May 1959	5.4	1.3	.188	*85	.15	278	
60-19a	Magma No. 3	do	do	21.9	*7.0	— 1949	—	6.0	—	—	—	—	SCS
	do	do	do	—	—	May 1959	10	3.8	—	*75	.03	49	
60-20	Riggs	Trib. Willcox Dry Lake	Dos Cabezas, Ariz.	.6	.6	— 1946	—	1.9	.099	—	—	—	SCS
	do	do	do	—	—	Oct. 1960	14	1.4	.073	*80	.06	105	
60-21	Upper Posto Wash	Posto Wash	Safford, Ariz.	4.95	4.95	— 1936	—	24.2	.209	—	—	—	SCS
	do	do	do	—	—	June 1958	22	19.5	.168	—	.04	—	
60-22	Whitlow Old Tank	Trib. of Gila River	Florence, Ariz.	.74	.74	— 1950	—	3.4	.254	—	—	—	SCS
	do	do	do	—	—	June 1960	10	1.8	.134	*85	.23	426	
60-23a	Bartlett	Verde	Sunflower, Ariz.	5,812	190	Feb. 1939	—	182,608	—	—	—	—	BR
	do	do	do	—	—	Aug. 1942	3.50	179,480	—	—	5/.154	—	
	do	do	do	—	—	Nov. 1950	8.33	7/179,568	—	—	—	—	
	do	do	do	—	—	Jan. 1964	13.17	178,488	—	—	.424	—	
60-24a	Horseshoe	do	do	5,618	5,614	Nov. 1945	—	67,900	.241	—	—	—	BR
	do	do	do	—	—	Oct. 1950	4.9	8/142,830	.507	—	.044	—	
	do	do	do	—	—	Nov. 1963	13.1	139,238	.496	—	.069	—	
60-25	Posto Wash Pond #3	Trib. of Posto Wash	Safford, Ariz.	.60	.58	— 1936*	—	19.10	*1.705	—	—	—	SCS
	do	do	do	—	—	Jan. 1964	*28	17.10	*1.527	76.2	.12	199	
60-26	Posto Wash Pond #4	Trib. of Gila River	do	.33	.33	— 1936*	—	6.462	*1.049	—	—	—	SCS
	do	do	do	—	—	Jan. 1964	*28	5.325	*.864	72.6	.12	190	
60-27	Williams-Chandler Pond #1	Trib. of Queen Creek	Florence Junction, Ariz.	1.09 *	1.09	— 1931	—	2.10	.197	—	—	—	SCS
	do	do	do	—	—	July 1961	30	1.00	.094	60	.033	43	
60-28	Williams-Chandler Pond #2	do	do	.65	.65	— 1944	—	9.33	1.366	—	—	—	SCS
	do	do	do	—	—	July 1961	17	8.95	1.291	60	.034	44	
60-29	Big Horn Mt. Tank #1	Trib. of Centennial Wash	Tonopah, Ariz.	.51	.51	June 1960	—	21.30	*7.83	—	—	—	SCS
	do	do	do	—	—	Oct. 1964	4.3	21.06	*7.74	47.6	.11	114	
60-30	Big Horn Mt. Tank #2	do	do	.44	.44	July 1960	—	11.46	*4.88	—	—	—	SCS
	do	do	do	—	—	Oct. 1964	4.3	11.26	*4.79	*50	.11	120	
60-31	Centennial Wash Tank	do	do	9/.60	9/.60	— 1954*	—	1.27	.397	—	—	—	SCS
	do	do	do	—	—	May 1965	*11	.67	.209	51.97	.09	180	
60-32	New Tank	do	do	9/1.88	9/1.88	— 1945	—	7.61	.759	—	—	—	SCS
	do	do	do	—	—	May 1965	20	4.08	.407	77.12	.096	158	
60-33	Wash Tank	do	Salome, Ariz.	.29	.29	June 1960	—	10.09	*6.51	—	—	—	SCS
	do	do	do	—	—	Oct. 1964	4.3	9.93	*6.41	52.5	.13	148.7	
60-34	Upper Twin Tank	Tiger Wash	Aguila, Ariz.	1.83	1.83	Oct. 1944*	—	2.21	*.228	—	—	—	SCS
	do	do	do	—	—	Oct. 1964	20	10/1.68	*.173	46	.015	15	
60-35	Harquahala Mt. Tank #1	Trib. of Brown's Canyon Wash	do	.11	.11	June 1958	—	6.64	*11.25	—	—	—	SCS
	do	do	do	—	—	Oct. 1964	6.3	6.50	*11.02	*50	.18	196	
60-36	Judia Wash Retarding Res.	Judia Wash	Solomon, Ariz.	4.60	4.57	Jan. 1957	—	158.79	*1.849	—	—	—	SCS
	do	do	do	—	—	Jan. 1964	7	154.24	*1.796	76.9	.14	234.5	
	do	do	do	—	—	July 1964	.6	153.16	*1.784	*76.9	.39	653.2	

60-37a	Three Bar B Debris Basin	Trib. of Rock Creek	Roosevelt, Ariz.	.073	.073	Aug.	1959	—	11/.23	.05	—	—	—	FS
	do	do	do	—	—	Oct.	1960	1.17	—	—	*110	12/18.77	12/44,969	
	do	do	do	—	—	Oct.	1961	1.00	—	—	*110	12/4.66	12/11,159	
	do	do	do	—	—	Oct.	1962	1.00	—	—	*110	0	0	
	do	do	do	—	—	Oct.	1964	2.00	—	—	*110	12/.55	12/1,313	
	do	do	do	—	—	Oct.	1965	1.00	—	—	*110	12/.14	12/328	
	do	do	do	—	—	Oct.	1966	1.00	—	—	*110	12/.27	12/656	
	do	do	do	—	—	Oct.	1967	1.00	—	—	*110	0	0	
	do	do	do	—	—	Oct.	1968	1.00	—	—	*110	12/.14	12/328	
	do	do	do	—	—	Oct.	1969	1.00	—	—	*110	0	0	
	do	do	do	—	—	Oct.	1970	1.00	—	—	*110	0	0	
60-38a	Three Bar C Debris Basin	do	do	.149	.149	Aug.	1959	—	11/.08	.002	—	—	—	FS
	do	do	do	—	—	Oct.	1960	1.17	—	—	*110	12/5.57	12/13,345	
	do	do	do	—	—	Oct.	1961	1.00	—	—	*110	12/.67	12/1,608	
	do	do	do	—	—	Oct.	1962	1.00	—	—	*110	12/.67	12/1,608	
	do	do	do	—	—	Oct.	1964	2.00	—	—	*110	0	0	
	do	do	do	—	—	Oct.	1965	1.00	—	—	*110	0	0	
	do	do	do	—	—	Oct.	1966	1.00	—	—	*110	12/.34	12/814	
	do	do	do	—	—	Oct.	1967	1.00	—	—	*110	0	0	
	do	do	do	—	—	Oct.	1968	1.00	—	—	*110	0	0	
	do	do	do	—	—	Oct.	1969	1.00	—	—	*110	0	0	
	do	do	do	—	—	Oct.	1970	1.00	—	—	*110	0	0	
60-39a	Three Bar D Debris Basin	do	do	.126	.126	Aug.	1959	—	11/.21	.01	—	—	—	FS
	do	do	do	—	—	Oct.	1960	1.17	—	—	*110	12/12.70	12/30,427	
	do	do	do	—	—	Oct.	1961	1.00	—	—	*110	12/3.73	12/8,937	
	do	do	do	—	—	Oct.	1962	1.00	—	—	*110	12/.63	12/1,521	
	do	do	do	—	—	Oct.	1964	2.00	—	—	*110	12/.04	12/95	
	do	do	do	—	—	Oct.	1965	1.00	—	—	*110	12/.08	12/190	
	do	do	do	—	—	Oct.	1966	1.00	—	—	*110	12/.08	12/190	
	do	do	do	—	—	Oct.	1967	1.00	—	—	*110	0	0	
	do	do	do	—	—	Oct.	1968	1.00	—	—	*110	0	0	
	do	do	do	—	—	Oct.	1969	1.00	—	—	*110	0	0	
	do	do	do	—	—	Oct.	1970	1.00	—	—	*110	0	0	
60-40a	Three Bar F Debris Basin	do	do	.107	.107	Mar.	1963	—	11/.04	.004	—	—	—	FS
	do	do	do	—	—	Oct.	1963	.51	—	—	*110	0	0	
	do	do	do	—	—	Oct.	1964	1.0	—	—	*110	12/.37	12/886	
	do	do	do	—	—	Oct.	1965	1.0	—	—	*110	12/.09	12/224	
	do	do	do	—	—	Oct.	1966	1.0	—	—	*110	12/.19	12/448	
	do	do	do	—	—	Oct.	1967	1.0	—	—	*110	0	0	
	do	do	do	—	—	Oct.	1968	1.0	—	—	*110	0	0	
	do	do	do	—	—	Oct.	1969	1.0	—	—	*110	0	0	
	do	do	do	—	—	Oct.	1970	1.0	—	—	*110	13/.08	13/2,016	
60-41	Bennett 1A	Walnut Gulch	Tombstone, Ariz.	.6	.6	June	1961	—	18	.6	—	—	—	SCS
	do	do	do	—	—	June	1967	6	17	.6	75	.2	326	
60-42	Cowan 7	do	do	.4	.4	June	1963	—	14	1.7	—	—	—	SCS
	do	do	do	—	—	June	1967	4	13.5	1.7	75	.25	410	
60-43	Horsethief Basin	Horsethief Canyon	Crown King, Ariz.	.85	.85	—	1934	—	42.4	.44	—	—	—	FS
	do	do	do	—	—	Nov.	1969	36	35.6	.37	—	.22	—	
60-44	Lynx Lake	Lynx Creek	Prescott, Ariz.	18.14	*10	Nov.	1962	—	1,472	1.07	—	—	—	FS
	do	do	do	—	—	Nov.	1968	6	1,460	1.06	—	.197	—	
60-45	Granite Basin	Mint Wash	do	4.69	*4.69	—	1939	—	89	*.32	—	—	—	FS
	do	do	do	—	—	Aug.	1967	28	58	*.21	—	.236	—	
60-46	Keller Tank	Trib. Verde River	Scottsdale, Ariz.	.06	.06	Apr.	1941	—	—	—	—	—	—	SCS
	do	do	do	—	—	Aug.	1970	29.2	1.529	*1.91	—	.09	—	
60-47	Gulleta Tank	Trib. Cont. Wash	Tonopah, Ariz.	.66	.645	June	1939	—	—	—	—	—	—	SCS
	do	do	do	—	—	Apr.	1967	27.9	21	6.63	74.8	.21	342	
60-48	Wallick Tank	Trib. Flying "E" Wash	Wickenburg, Ariz.	.79	.77	May	1956	—	79.1	20.87	—	—	—	SCS
	do	do	do	—	—	Sept.	1970	14.3	77.3	20.39	94.1	.16	328	
60-49	Pasquette Tank	Trib. Cemetery Wash	do	.13	.129	May	1965	—	.99	1.59	—	—	—	SCS
	do	do	do	—	—	Dec.	1970	5.6	.91	1.46	88.2	.10	192	

LITTLE COLORADO AND SAN JUAN RIVER BASINS

61-1	Oak Creek Reservoir	Trib. of Zuni River	Zuni, N. Mex.	9.41	9.41	Oct.	1954	—	334.8	1.625	—	—	—	GS
	do	do	do	—	—	May	1960	14/6	148.0	.630	—	3.32	—	

1/ Drainage area includes 1.87 sq. mi. diverted from Hash Hollow.
2/ Reservoir was full of sediment in 1954.
3/ Constructed 1943, cleaned 1953.
4/ Constructed 1941, cleaned 1952.
5/ This reservoir is downstream from Upper Fenta Wash Reservoir. Little or no sediment is contributed from the 4.17 sq. mi. above this dam.
6/ Based on total drainage area of 5,812 sq. mi. before construction of Horseshoe Dam upstream.
7/ Increase in capacity probably due to compaction.
8/ Capacity increased 76,130 ac.-ft. by installation of spillway gates in June 1950.
9/ Drainage area is smallest area that could be positively delineated.
10/ Storage capacity at original spillway crest elevation - storage capacity at eroded spillway elevation 0.53 ac.-ft.
11/ Basin cleaned as needed to maintain capacity.
12/ Includes sediment trapped above spillway level.
13/ Basin filled during storm of 9-5-70 and undetermined amount of sediment escaped from pond.
14/ Runoff seasons.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES) TOTAL	DRAINAGE AREA (SQUARE MILES) NET	DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN AC.-FT.	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN TONS	AGENCY SUPPLYING DATA
			LITTLE COLORADO AND SAN JUAN RIVER BASINS (Continued)										
61-2a	Millet Swale Reservoir	Trib. of Silver Creek	Taylor, Ariz.	40.0	39.7	June 1959	—	1,342.0	2.1	—	—	—	SCS
	do	do	do	—	—	June 1962	3	1,259.4	2.0	67	0.69	1,007	
	do	do	do	—	—	June 1966	4	1,118.5	1.7	80	.89	1,551	
61-3	Stockyard Tank	Trib. Cottonwood Wash	Snowflake, Ariz.	.48	.476	— 1943	—	6.04	*.79	—	—	—	SCS
	do	do	do	—	—	May 1968	25	4.02	*.52	101.6	.17	376	
61-4	Thomas Tank I	do	do	.55	.549	— 1947	—	3.09	*.35	—	—	—	SCS
	do	do	do	—	—	June 1968	21	1.38	*.16	77.9	.15	254	
61-5	Ellsworth Tank	Trib. Show Low Creek	Taylor, Ariz.	.25	.249	— 1948	—	1.50	*.23	—	—	—	SCS
	do	do	do	—	—	Aug. 1968	20	1.27	*.19	80	.044	79	
61-6	New Tank	Trib. Cottonwood Wash	Snowflake, Ariz.	2.45	2.45	— 1938	—	10.13	*.15	—	—	—	SCS
	do	do	do	—	—	May 1968	30	9.10	*.14	87.7	.014	27	
61-7	West Hall Tank	do	Clay Springs, Ariz.	.32	.32	— 1944	—	.76	*.09	—	—	—	SCS
	do	do	do	—	—	June 1968	24	.61	*.07	93.4	.02	41	
61-8	Thomas Tank II	Trib. Perkins Spring Draw	do	.10	.10	June 1964	—	1.39	*.52	—	—	—	SCS
	do	do	do	—	—	July 1968	4	1.35	*.50	89.3	.10	194	
			COLORADO RIVER BASIN (HALLS CROSSING TO HOOVER DAM)										
62-1	Lake Mead (Hoover Dam)	Colorado River	Boulder City, Nev.	167,800	1/167,600	Feb. 1935	—	31,250,000	2.140	—	—	—	GS
	do	do	do	—	—	Sept. 1948 2/	13.7	29,827,000	2.043	3/65	.620	877	
62-2	Brooksby Tank	Trib. of Johnson Wash	Fredonia, Ariz.	.4	.4	— 1951	—	.697	.327	—	—	—	SCS
	do	do	do	—	—	Sept. 1965	4/14	.140	.066	97	4/.10	211	
62-3	Riggs Flat Charco	Sandy Canyon Wash	do	57.86	57.83	Oct. 1956	—	54.93	.178	—	—	—	SCS
	do	do	do	—	—	Sept. 1965	8.9	36.58	.119	72	.035	55	
62-4	Leroy Judd Tank	Trib. of Kanab Creek	do	.38	.38	— 1962	—	.95	.468	—	—	—	SCS
	do	do	do	—	—	Aug. 1965	3	.22	.108	105.7	.63	1,450	
62-5	Kaibab Indian Reservation Erosion Control Structure.	do	Indian Moccasin, Ariz.	1.27	1.27	— 1935	—	6.008	.887	—	—	—	SCS
	do	do	do	—	—	Oct. 1965	30	2.267	.332	87.2	.098	186	
62-6	Jesse Judd Tank	do	Fredonia, Ariz.	5/3.49	5/3.49	— 1955	—	7.10	—	—	—	—	SCS
	do	do	do	—	—	Aug. 1965	6/10	2.77	—	*80	6/.12	209	
62-7	Frog Hollow Detention Res.	Frog Hollow Wash	Hurricane, Utah	9.2	9.2	Jan. 1957	—	223.0	*1.517	—	—	—	SCS
	do	do	do	—	—	Jan. 1958	*1	216.4	*1.472	*80	.72	1,255	
	do	do	do	—	—	Nov. 1965	7.8	140.8	*.958	*80	1.05	1,830	
62-8	CCC Pond (West)	Trib. Santa Clara River	St. George, Utah	.03	.03	— 1941	—	—	—	—	—	—	SCS
	do	do	do	—	—	Apr. 1966	25	*.2	*.63	92	7/.23	461	
62-9	CCC Pond (East)	do	do	.32	.32	— 1941	—	—	—	—	—	—	SCS
	do	do	do	—	—	Apr. 1966	25	*.2	*.06	90	7/.017	33	
62-10	[illegible] Debris Basin #2	Trib. Virgin River	do	.026	.026	— 1944	—	—	—	—	—	—	SCS
	do	do	do	—	—	Feb. 1966	22	*1.5	*5.3	*90	7/3.35	6,566	
62-11	[illegible] Debris Basin #1	do	do	.09	.09	— 1944	—	—	—	—	—	—	SCS
	do	do	do	—	—	Feb. 1966	22	*1.0	*1.0	*90	7/.61	1,196	
62-12	Iliff Andrus	Trib. Warner Draw	do	1.09	1.08	— 1949	—	—	—	—	—	—	SCS
	do	do	do	—	—	Jan. 1966	17	*20	*2.2	78	7/.76	1,290	
62-13	Winfred Spendlove	Trib. Fort Pearce Wash	do	.85	.84	— 1946	—	—	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1966	20	*7.0	*1.1	87.5	7/.42	800	
62-14	Black Knolls	Trib. City Creek	do	.59	.58	— 1922	—	—	—	—	—	—	SCS
	do	do	do	—	—	Nov. 1965	43	*20	*3.2	*75	.34	555	
62-15	Little Grassy Pond	Unknown	do	2.3	2.3	— 1943	—	—	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1966	23	*2.0	—	94	7/.043	88	
62-16	Bill Shaw Pond	Trib. Santa Clara River	do	.057	.057	— 1956	—	—	—	—	—	—	SCS
	do	do	do	—	—	June 1966	10	*2.0	*3.3	72	7/1.79	2,807	
62-17	Manganese Wash	Manganese Wash	do	.17	.17	— 1956	—	—	—	—	—	—	SCS
	do	do	do	—	—	Apr. 1966	10	*1.0	*.2	80	7/.38	662	
			COLORADO RIVER BASIN (ABOVE HALLS CROSSING) Gunnison, Dolores and Fremont River Basins										
63-1	Badger Detention (1A)	Badger Wash	Mack, Colo.	1.53	1.49	May 1957	—	201.52	3.336	—	—	—	GS
	do	do	do	—	—	Nov. 1958	1.6	199.35	3.300	—	.91	—	
	do	do	do	—	—	Nov. 1959	1.0	196.27	3.250	—	2.07	—	
	do	do	do	—	—	Nov. 1961	2.0	190.38	3.152	—	1.97	—	

63-2	Badger Wash West Twin (2A)	do.	do.	.167	.165	Dec. 1953	—	6.30	.957	—	—	—	88
	do.	do.	do.	—	—	July 1955	1.6	4.47	.679	—	6.91	—	
	do.	do.	do.	—	—	Nov. 1956	1.4	4.47	.679	—	0	—	
	do.	do.	do.	—	—	Oct. 1957	1.0	3.98	.605	—	2.97	—	
	do.	do.	do.	—	—	Nov. 1958	1.0	3.97	.605	—	0	—	
	do.	do.	do.	—	—	Nov. 1959	1.0	3.65	.555	—	2.00	—	
	do.	do.	do.	—	—	Nov. 1961	2.0	2.43	.369	—	3.70	—	
63-3	Badger Wash Lower Oil Well (3A)	do.	do.	.059	.055	Dec. 1953	—	12.90	4.813	—	—	—	88
	do.	do.	do.	—	—	July 1955	1.6	12.65	4.720	—	2.91	—	
	do.	do.	do.	—	—	Nov. 1956	1.4	12.65	4.720	—	0	—	
	do.	do.	do.	—	—	Oct. 1957	1.0	12.55	4.683	—	1.83	—	
	do.	do.	do.	—	—	Nov. 1958	1.0	12.55	4.683	—	0	—	
	do.	do.	do.	—	—	Nov. 1959	1.0	12.33	4.602	—	4.00	—	
	do.	do.	do.	—	—	Nov. 1961	2.0	12.12	4.522	—	1.83	—	
63-4	East Basin (11)	do.	do.	.089	.087	Apr. 1954	—	5.45	1.135	—	—	—	88
	do.	do.	do.	—	—	Nov. 1956	2.6	4.92	1.025	—	2.30	—	
	do.	do.	do.	—	—	Oct. 1957	1.0	4.76	.992	—	1.84	—	
	do.	do.	do.	—	—	Nov. 1959	2.0	4.65	.969	—	.69	—	
63-5	Middle Basin (12)	do.	do.	.092	.088	Apr. 1954	—	16.93	2.998	—	—	—	88
	do.	do.	do.	—	—	Nov. 1956	2.6	15.87	2.809	—	4.66	—	
	do.	do.	do.	—	—	Oct. 1957	1.0	15.38	2.722	—	5.97	—	
	do.	do.	do.	—	—	Nov. 1959	2.0	15.32	2.712	—	.34	—	
63-6a	Prairie Dog (4A)	Trib. Badger Wash	do.	.022	.022	Dec. 1953	—	3.05	4.36	—	—	—	88
	do.	do.	do.	—	—	July 1955	1.6	2.64	3.77	e90	11.8	23,130	
	do.	do.	do.	—	—	Nov. 1956	1.3	2.64	3.77	e90	0	—	
	do.	do.	do.	—	—	Oct. 1957	1.0	2.49	3.56	e90	6.82	13,365	
	do.	do.	do.	—	—	Nov. 1958	1.1	2.49	3.56	e90	0	—	
	do.	do.	do.	—	—	Nov. 1959	1.0	2.43	3.47	e90	2.73	5,346	
	do.	do.	do.	—	—	Nov. 1961	2.0	2.28	3.26	e90	3.18	6,082	
	do.	do.	do.	—	—	Nov. 1963	2.0	2.17	3.10	e90	2.27	4,900	
	do.	do.	do.	—	—	Nov. 1964	1.0	2.03	2.90	e90	6.36	12,476	
	do.	do.	do.	—	—	Nov. 1965	1.0	1.82	2.60	e90	9.55	18,711	
	do.	do.	do.	—	—	Nov. 1966	1.0	1.91	2.73	e90	—	—	
	do.	do.	do.	—	—	Oct. 1967	1.0	1.80	2.57	e90	5.30	9,002	
	do.	do.	do.	—	—	Nov. 1968	1.0	1.66	2.37	e90	6.36	10,496	
	do.	do.	do.	—	—	June 1970	1.6	8/2.52	3.60	e90	0	—	
	do.	do.	do.	—	—	Oct. 1970	.4	2.50	3.57	e90	2.27	4,455	
63-7	Southeast (13)	Badger Wash	do.	.484	.477	Apr. 1954	—	27.25	1.854	—	—	—	88
	do.	do.	do.	—	—	Nov. 1956	2.6	24.85	1.690	—	1.93	—	
	do.	do.	do.	—	—	Oct. 1957	1.0	23.44	1.595	—	2.96	—	
	do.	do.	do.	—	—	Nov. 1959	2.0	22.26	1.514	—	1.26	—	
63-8a	Upper Basin (1A)	Trib. Badger Wash	do.	.066	.066	Dec. 1953	—	8.30	3.81	—	—	—	88
	do.	do.	do.	—	—	July 1955	1.6	7.59	3.48	e90	6.68	13,096	
	do.	do.	do.	—	—	Nov. 1956	1.3	7.59	2.96	e90	0	—	
	do.	do.	do.	—	—	Nov. 1957	1.0	7.34	3.37	e90	3.78	7,425	
	do.	do.	do.	—	—	Nov. 1958	1.1	7.34	3.37	e90	0	—	
	do.	do.	do.	—	—	Nov. 1959	1.0	7.06	3.23	e90	4.55	9,061	
	do.	do.	do.	—	—	Nov. 1961	2.0	6.70	3.07	e90	2.73	5,198	
	do.	do.	do.	—	—	Nov. 1963	2.0	6.59	3.02	e90	.83	1,636	
	do.	do.	do.	—	—	Nov. 1964	1.0	6.55	3.00	e90	.60	1,188	
	do.	do.	do.	—	—	Nov. 1965	1.0	6.10	2.86	e90	6.82	13,365	
	do.	do.	do.	—	—	Nov. 1966	1.0	6.16	2.82	e90	9/—	—	
	do.	do.	do.	—	—	Oct. 1967	1.0	6.00	2.73	e90	2.41	4,752	
	do.	do.	do.	—	—	Nov. 1968	1.0	5.48	2.51	e90	7.86	15,444	
	do.	do.	do.	—	—	Nov. 1969	1.0	5.54	2.54	e90	9/—	—	
63-9	West Twin (2-1)	do.	do.	.148	.148	Dec. 1953	—	6.30	1.31	—	—	—	88
	do.	do.	do.	—	—	July 1955	1.6	4.38	.91	e90	8.10	15,878	
	do.	do.	do.	—	—	Nov. 1956	1.3	4.38	.91	e90	0	—	
	do.	do.	do.	—	—	Oct. 1957	1.0	3.90	.81	e90	3.24	6,357	
	do.	do.	do.	—	—	Nov. 1958	1.1	3.90	.81	e90	0	—	
	do.	do.	do.	—	—	Nov. 1959	1.0	10/15.08	3.13	e90	1.49	2,924	
	do.	do.	do.	—	—	Nov. 1961	2.0	13.86	2.88	e90	4.18	8,079	
	do.	do.	do.	—	—	Nov. 1963	2.0	13.52	2.80	e90	1.15	2,253	
	do.	do.	do.	—	—	Nov. 1964	1.0	12.39	2.57	e90	7.64	14,966	
	do.	do.	do.	—	—	Nov. 1965	1.0	12.06	2.50	e90	2.23	4,371	
	do.	do.	do.	—	—	Nov. 1966	1.0	12.10	2.51	e90	9/—	—	
	do.	do.	do.	—	—	Oct. 1967	1.0	11.53	2.39	e90	3.85	7,549	
	do.	do.	do.	—	—	Nov. 1968	1.0	10.96	2.09	e90	6.55	12,847	
	do.	do.	do.	—	—	Nov. 1969	1.0	10.64	2.10	e90	9/—	—	

1/ Not adjusted for numerous small reservoirs. Runoff used to compute C/I ratio was not adjusted for upstream diversion.

2/ Period of survey was from Mar. 1948 - Mar. 1949.

3/ Based on measured and estimated sediment inflow of 2,000,000,000 tons in 13.7 yr. period.

4/ Estimated effective life of pond is 5 yr. 1951 - 1956.

5/ Reservoir is off main channel. Estimate reservoir intercepts 1/3 of flow and sediment from total drainage area.

6/ Effective life of reservoir is estimated at 8 yr. storm in 1963 filled reservoir and changed channel course.

7/ Includes deposition above spillway crest level.

8/ Reservoir cleaned in June 1970.

9/ Capacity increase.

10/ Dam and spillway raised in 1959.

e Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE -FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
				COLORADO RIVER BASIN (ABOVE HALLS CROSSING) Gunnison, Dolores and Fremont River Basins (Continued)									
63-10	Oil Well (3-4)	Trib. of Badger Wash	Mack, Colo.	0.059	0.059	Dec. 1953	—	12.92	5.71	—	—	—	GS
	do	do	do	—	—	July 1955	1.6	12.18	5.39	*90	7.80	15,289	
	do	do	do	—	—	Nov. 1956	1.3	12.18	5.39	*90	0	—	
	do	do	do	—	—	Oct. 1957	1.0	11.90	5.26	*90	4.75	9,302	
	do	do	do	—	—	Nov. 1958	1.1	11.90	5.26	*90	0	—	
	do	do	do	—	—	Nov. 1959	1.0	11.64	5.15	*90	4.40	8,630	
	do	do	do	—	—	Nov. 1961	2.0	11.31	5.00	*90	2.71	5,482	
	do	do	do	—	—	Nov. 1963	2.0	11.25	4.98	*90	.51	997	
	do	do	do	—	—	Nov. 1964	1.0	11.21	4.96	*90	.68	1,329	
	do	do	do	—	—	Nov. 1965	1.0	10.73	4.75	*90	8.14	15,967	
	do	do	do	—	—	Nov. 1966	1.0	10.82	4.79	*90	1/—	—	
	do	do	do	—	—	Oct. 1967	1.0	10.62	4.70	*90	3.39	6,645	
	do	do	do	—	—	Nov. 1968	1.0	10.15	4.49	*90	7.96	15,615	
	do	do	do	—	—	Nov. 1969	1.0	10.13	4.48	*90	.34	664	
	do	do	do	—	—	Oct. 1970	1.0	9.99	4.42	*90	2.38	4,651	
63-11	Windy Point (4-8)	do	do	.019	.019	Dec. 1953	—	4.52	9.62	—	--	—	GS
	do	do	do	—	—	July 1955	1.6	4.31	9.17	*90	6.85	13,427	
	do	do	do	—	—	Nov. 1956	1.3	4.31	9.17	*90	0	—	
	do	do	do	—	—	Oct. 1957	1.0	4.19	8.91	*90	6.32	12,380	
	do	do	do	—	—	Nov. 1958	1.1	4.19	8.91	*90	0	—	
	do	do	do	—	—	Nov. 1959	1.0	4.16	8.85	*90	1.58	3,095	
	do	do	do	—	—	Nov. 1961	2.0	4.14	8.80	*90	.53	1,032	
	do	do	do	—	—	Nov. 1963	2.0	4.12	8.76	*90	.53	1,032	
	do	do	do	—	—	Nov. 1964	1.0	4.03	8.57	*90	4.74	9,285	
	do	do	do	—	—	Nov. 1965	1.0	3.89	8.28	*90	7.37	14,443	
	do	do	do	—	—	Nov. 1966	1.0	3.89	8.28	*90	0	—	
	do	do	do	—	—	Oct. 1967	1.0	3.81	8.11	*90	4.21	8,253	
	do	do	do	—	—	Nov. 1968	1.0	3.74	7.96	*90	3.68	7,222	
	do	do	do	—	—	Nov. 1969	1.0	3.83	8.15	*90	1/—	—	
63-12	Yucca (2-8)	do	do	.158	.158	Dec. 1953	—	8.45	2.14	—	—	—	GS
	do	do	do	—	—	July 1955	1.6	6.05	1.53	*90	9.50	18,622	
	do	do	do	—	—	Nov. 1956	1.3	6.05	1.53	*90	0	—	
	do	do	do	—	—	Oct. 1957	1.0	5.94	1.50	*90	.70	1,365	
	do	do	do	—	—	Nov. 1958	1.1	5.94	1.50	*90	0	—	
	do	do	do	—	—	Nov. 1959	1.0	2/24.80	6.28	*90	2.15	2,840	
	do	do	do	—	—	Nov. 1961	2.0	24.18	6.12	*90	1.96	3,846	
	do	do	do	—	—	Nov. 1963	2.0	23.89	6.05	*90	.95	1,110	
	do	do	do	—	—	Nov. 1964	1.0	23.45	5.94	*90	2.78	5,459	
	do	do	do	—	—	Nov. 1965	1.0	23.15	5.86	*90	1.90	3,722	
	do	do	do	—	—	Nov. 1966	1.0	23.23	5.88	*90	1/—	—	
	do	do	do	—	—	Oct. 1967	1.0	22.86	5.79	*90	2.34	4,590	
	do	do	do	—	—	Nov. 1968	1.0	21.88	5.54	*90	6.20	12,158	
	do	do	do	—	—	Nov. 1969	1.0	22.12	5.60	*90	1/—	—	
63-13	North Basin (3-8)	do	do	.048	.048	Dec. 1953	—	8.10	4.60	—	—	—	GS
	do	do	do	—	—	July 1955	1.6	7.69	4.37	*90	5.42	10,626	
	do	do	do	—	—	Nov. 1956	1.3	7.69	4.37	*90	0	—	
	do	do	do	—	—	Oct. 1957	1.0	7.58	4.31	*90	2.29	4,492	
	do	do	do	—	—	Nov. 1958	1.1	7.58	4.31	*90	0	—	
	do	do	do	—	—	Nov. 1959	1.0	7.51	4.27	*90	1.46	2,859	
	do	do	do	—	—	Nov. 1961	2.0	7.38	4.19	*90	1.46	2,654	
	do	do	do	—	—	Nov. 1963	2.0	7.10	4.03	*90	2.92	5,717	
	do	do	do	—	—	Nov. 1964	1.0	7.05	4.00	*90	1.04	2,042	
	do	do	do	—	—	Nov. 1965	1.0	6.81	3.87	*90	5.00	9,801	
	do	do	do	—	—	Nov. 1966	1.0	6.81	3.87	*90	0	—	
	do	do	do	—	—	Oct. 1967	1.0	6.54	3.72	*90	5.63	11,026	
	do	do	do	—	—	Nov. 1968	1.0	6.07	3.45	*90	9.80	19,196	
	do	do	do	—	—	Nov. 1969	1.0	6.18	3.51	*90	1/—	—	
	do	do	do	—	—	Oct. 1970	1.0	6.13	3.48	*90	1.04	2,042	

63-14	Lower Banks (1-8)	do	do	.084	.084	Dec.	1953	—	19.80	8.69	—	—	—	GS
	do	do	do	—	—	July	1955	1.6	19.17	8.41	*90	4.65	9,095	
	do	do	do	—	—	Nov.	1956	1.3	19.17	8.41	*90	0	0	
	do	do	do	—	—	Oct.	1957	1.0	19.17	8.41	*90	0	0	
	do	do	do	—	—	Nov.	1958	1.1	19.17	8.41	*90	0	0	
	do	do	do	—	—	Nov.	1959	1.0	19.17	8.41	*90	0	0	
	do	do	do	—	—	Nov.	1961	2.0	18.85	8.27	*90	1.90	3,734	
	do	do	do	—	—	Nov.	1963	2.0	18.74	8.22	*90	.79	1,283	
	do	do	do	—	—	Nov.	1964	1.0	18.72	8.21	*90	.24	467	
	do	do	do	—	—	Nov.	1965	1.0	18.38	8.06	*90	4.05	7,934	
	do	do	do	—	—	Nov.	1966	1.0	18.42	8.08	*90	1/—	—	
	do	do	do	—	—	Oct.	1967	1.0	18.25	8.00	*90	2.02	3,957	
	do	do	do	—	—	Nov.	1968	1.0	17.63	7.73	*90	7.38	14,468	
	do	do	do	—	—	Nov.	1969	1.0	17.60	7.72	*90	.36	700	
63-15	Bass Mountain No.1	Trib. Alkali Creek	Wolcott, Colo.	.0116	.011	July	1965	—	1.877	—	—	—	—	GS
	do	do	do	—	—	Nov.	1965	.30	1.851	2.44	*90	7.91	15,500	
	do	do	do	—	—	Oct.	1966	1.0	1.860	2.46	*90	1/—	—	
	do	do	do	—	—	Nov.	1967	1.0	1.852	2.45	*90	.73	1,420	
	do	do	do	—	—	Nov.	1968	1.0	1.842	2.43	*90	.91	1,780	
	do	do	do	—	—	Nov.	1969	1.0	1.841	2.43	*90	.09	176	
	do	do	do	—	—	Nov.	1970	1.0	1.837	2.42	*90	.36	710	
63-16	Bass Mountain No. 3	do	do	.008	.008	July	1965	—	1.686	3.24	—	—	—	GS
	do	do	do	—	—	Nov.	1965	.28	1.644	3.16	*90	18.8	36,850	
	do	do	do	—	—	Oct.	1966	1.0	1.651	3.18	*90	1/—	—	
	do	do	do	—	—	Nov.	1967	1.0	1.634	3.14	*90	2.13	4,165	
	do	do	do	—	—	Nov.	1968	1.0	1.637	3.15	*90	1/—	—	
	do	do	do	—	—	Nov.	1969	1.0	1.619	3.11	*90	2.25	4,410	
	do	do	do	—	—	Nov.	1970	1.0	1.613	3.10	*90	.75	1,470	
63-17	Fool Creek Debris Basin	Fool Creek	Fraser, Colo.	1.11	1.11	Oct.	1951	—	—	—	—	—	—	FS
	do	do	do	—	—	Oct.	1952	1.0	—	—	—	.032	65.28	
	do	do	do	—	—	Oct.	1953	1.0	—	—	—	.016	32.64	
	do	do	do	—	—	Oct.	1954	1.0	—	—	—	—	0	
	do	do	do	—	—	Oct.	1955	1.0	—	—	—	—	0	
	do	do	do	—	—	Oct.	1956	1.0	—	—	—	.026	53.12	
	do	do	do	—	—	Oct.	1957	1.0	—	—	—	.051	101.76	
	do	do	do	—	—	Oct.	1958	1.0	—	—	—	.031	62.08	
	do	do	do	—	—	Oct.	1959	1.0	—	—	—	.006	12.48	
	do	do	do	—	—	Oct.	1960	1.0	—	—	—	.010	20.16	
	do	do	do	—	—	Oct.	1961	1.0	—	—	—	.004	8.96	
	do	do	do	—	—	Oct.	1962	1.0	—	—	—	.012	23.68	
	do	do	do	—	—	Oct.	1963	1.0	—	—	—	—	0	
	do	do	do	—	—	Oct.	1964	1.0	—	—	—	—	0	
	do	do	do	—	—	Oct.	1965	1.0	—	—	—	.018	36.16	
	do	do	do	—	—	Oct.	1966	1.0	—	—	—	.004	8.0	
	do	do	do	—	—	Oct.	1967	1.0	—	—	—	.005	10.32	
	do	do	do	—	—	Oct.	1968	1.0	—	—	—	.008	17.96	
	do	do	do	—	—	Oct.	1969	1.0	—	—	—	.010	22.45	
	do	do	do	—	—	Oct.	1970	1.0	—	—	—	.011	24.44	
63-18	Deadhorse Weir	Deadhorse Creek	do	1.04	1.04	Oct.	1954	—	—	—	—	—	—	FS
	do	do	do	—	—	Oct.	1955	1.0	—	—	—	.003	6.4	
	do	do	do	—	—	Oct.	1956	1.0	—	—	—	.012	26.88	
	do	do	do	—	—	Oct.	1957	1.0	—	—	—	.016	33.52	
	do	do	do	—	—	Oct.	1958	1.0	—	—	—	.006	3.2	
	do	do	do	—	—	Oct.	1959	1.0	—	—	—	.004	7.68	
	do	do	do	—	—	Oct.	1960	1.0	—	—	—	.004	7.68	
	do	do	do	—	—	Oct.	1961	1.0	—	—	—	.001	1.60	
	do	do	do	—	—	Oct.	1962	1.0	—	—	—	.009	19.84	
	do	do	do	—	—	Oct.	1963	1.0	—	—	—	.001	1.28	
	do	do	do	—	—	Oct.	1964	1.0	—	—	—	.001	2.24	
	do	do	do	—	—	Oct.	1965	1.0	—	—	—	.005	11.20	
	do	do	do	—	—	Oct.	1966	1.0	—	—	—	.001	1.28	
	do	do	do	—	—	Oct.	1967	1.0	—	—	—	.0004	.83	
	do	do	do	—	—	Oct.	1968	1.0	—	—	—	.001	1.81	
	do	do	do	—	—	Oct.	1969	1.0	—	—	—	.002	3.46	
	do	do	do	—	—	Oct.	1970	1.0	—	—	—	.005	10.71	

1/ Capacity increase.
2/ Dam and spillway raised in 1959.

* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
COLORADO RIVER BASIN (ABOVE HALLS CROSSING) Gunnison, Dolores and Fremont River Basins (Continued)													
63-19	Lexen Creek Weir	Lexen Creek	Fraser, Colo.	0.48	0.48	Oct. 1955	—	—	—	—	—	—	FS
	do	do	do	—	—	Oct. 1956	1.0	—	—	—	0.005	10.24	
	do	do	do	—	—	Oct. 1957	1.0	—	—	—	.009	17.92	
	do	do	do	—	—	Oct. 1958	1.0	—	—	—	.011	21.12	
	do	do	do	—	—	Oct. 1959	1.0	—	—	—	.002	3.84	
	do	do	do	—	—	Oct. 1960	1.0	—	—	—	.002	3.52	
	do	do	do	—	—	Oct. 1961	1.0	—	—	—	.0003	.64	
	do	do	do	—	—	Oct. 1962	1.0	—	—	—	.005	8.96	
	do	do	do	—	—	Oct. 1963	1.0	—	—	—	.0002	.32	
	do	do	do	—	—	Oct. 1964	1.0	—	—	—	.002	2.88	
	do	do	do	—	—	Oct. 1965	1.0	—	—	—	.003	5.76	
	do	do	do	—	—	Oct. 1966	1.0	—	—	—	.002	2.24	
	do	do	do	—	—	Oct. 1967	1.0	—	—	—	.001	1.73	
	do	do	do	—	—	Oct. 1968	1.0	—	—	—	.001	1.50	
	do	do	do	—	—	Oct. 1969	1.0	—	—	—	.001	1.50	
	do	do	do	—	—	Oct. 1970	1.0	—	—	—	.002	4.00	
63-20	East St. Louis Creek Weir	East St. Louis Creek	do	3.10	3.10	Oct. 1964	1.0	—	—	—	—	—	FS
	do	do	do	—	—	Oct. 1965	1.0	—	—	*100	.002	5.45	
	do	do	do	—	—	Oct. 1966	1.0	—	—	*100	.001	1.35	
	do	do	do	—	—	Oct. 1967	1.0	—	—	*100	.002	3.40	
	do	do	do	—	—	Oct. 1968	1.0	—	—	*100	.002	4.20	
	do	do	do	—	—	Oct. 1969	1.0	—	—	*100	.001	2.90	
	do	do	do	—	—	Oct. 1970	1.0	—	—	*100	.002	4.25	
63-21	MW-1 Roatcap Wash Watershed	Roatcap Wash	Olathe, Colo.	11.6	11.5	Dec. 1964	—	829.6	*7.0	—	—	—	SCS
	do	do	do	—	—	Nov. 1970	6	820.0	*6.9	—	.28	—	
GREEN RIVER BASIN													
64-1	Duck Fork	Ferron Creek	Ferron, Utah	3.56	3.56	— 1942	—	718	*.234	—	—	—	SCS
	do	do	do	—	—	Oct. 1962	20	1/870	*.286	*60	.14	183	
64-2	South Soda Creek Weir	South Soda Creek	Steamboat Springs, Colo.	3.40	3.40	Oct. 1967	—	—	—	—	—	—	FS
	do	do	do	—	—	Oct. 1968	1.0	—	—	*135	.005	13.43	
	do	do	do	—	—	Oct. 1969	1.0	—	—	*135	.0003	1.08	
	do	do	do	—	—	Oct. 1970	1.0	—	—	*135	.005	14.92	
64-3	North Fish Creek Weir	North Fish Creek	do	2.24	—	Oct. 1968	—	—	—	—	—	—	FS
	do	do	do	—	—	Oct. 1969	1.0	—	—	*135	.001	1.62	
	do	do	do	—	—	Oct. 1970	1.0	—	—	*135	.004	11.74	
64-4	West Walton Creek Weir	Trib. Walton Creek	do	1.33	1.33	Oct. 1967	—	—	—	—	—	—	FS
	do	do	do	—	—	Oct. 1968	1.0	—	—	*110	.016	38.28	
	do	do	do	—	—	Oct. 1969	1.0	—	—	*110	.006	15.07	
	do	do	do	—	—	Oct. 1970	1.0	—	—	*110	.020	47.90	
64-5	Bullwinkle	Trib. Brush Creek	Vernal, Utah	.11	.11	— 1938	—	—	—	—	—	—	SCS
	do	do	do	—	—	Oct. 1968	30	*5.0	—	*70	2/.79	1,204	
64-6	Rock Canyon	Rock Canyon	do	1.41	1.41	— 1936	—	—	—	—	—	—	SCS
	do	do	do	—	—	Oct. 1968	22	*5.0	—	*60	.011	14	
64-7	Wiles Haslem Pond	Trib. Steinaker Draw	do	.095	.095	— 1952	—	—	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1967	15	*2.0	—	*70	2/.67	1,021	
64-8	Masser Reservoir	Trib. Twelve Mile Wash	do	.65	.65	— 1942	—	—	—	—	—	—	SCS
	do	do	do	—	—	May 1968	26	*8.0	—	*90	2/.23	450	
GREAT SALT LAKE BASIN													
65-1	East Canyon	East Canyon Creek	Morgan, Utah	144	144	— 1896	—	3/3,850	.098	—	—	—	BR
	do	do	do	—	—	Oct. 1954	58	28,730	.733	4/77.1	.124	208	
65-2	Echo	Weber River	Echo, Utah	732	732	Oct. 1930	—	75,718	.373	—	—	—	BR
	do	do	do	—	—	Oct. 1954	24	73,900	.364	4/71	.104	161	
65-3a	Santaquin Debris Basin	Summit Creek	Santaquin, Utah	17.65	13.5	Fall 1954	—	*84	.008	—	—	—	SCS
	do	do	do	—	—	Dec. 1957	3	*65	.006	*90	.47	921	
SEVIER RIVER BASIN													
66-1	Rocky Hale	Rocky Hale Creek	Koosharem, Utah	5.0	4.9	— 1895	—	609	—	—	—	—	SCS
	do	do	do	—	—	Oct. 1960	75	598	—	—	.024	—	

66-2	Indian Creek No. 1	Indian Creek	Beaver, Utah	12.0	11.95	—	1898	—	318	—	—	—	—	SCS
	do	do	do	—	—	Nov.	1940	42	299	—	—	.038	—	
66-3	Rocky Ford	Beaver River	do	510	508	Feb.	1915	—	23,260	—	—	—	—	SCS
	do	do	do	—	—	Nov.	1940	25.8	21,509	—	—	.134	—	
66-4	Rocky Ford	Sevier River	Sigurd, Utah	900	900	—	1890	—	2,115	—	—	—	—	SCS
	do	do	do	—	—	Nov.	1940	50	790	—	—	.029	—	
66-5	Skutumpah	Skutumpah Creek	Salina, Utah	10.0	9.92	—	1893	—	667	—	—	—	—	SCS
	do	do	do	—	—	Nov.	1940	47	430	—	—	.508	—	
66-6	Enterprise	Pine Creek	Enterprise, Utah	25.0	23.4	—	1909	—	9,000	—	—	—	—	SCS
	do	do	do	—	—	Nov.	1940	31	8,550	—	—	.620	—	
66-7	Yankee Meadows	Pole Creek	Parowan, Utah	7.0	6.9	Dec.	1926	—	2,500	—	—	—	—	SCS
	do	do	do	—	—	Nov.	1940	14	2,200	—	—	3.10	—	
66-8	Piute	Sevier River	Marysville, Utah	2,440	2,436	—	1910[5]	—	81,200	—	—	—	—	SCS
	do	do	do	—	—	—	1938	28	74,010	—	—	.106	—	
66-9	Sevier Bridge	do	Nephi, Utah	5,120	1,089	—	1908	—	250,000	—	—	—	—	SCS
	do	do	do	—	—	—	1932	24	234,462	—	—	.596	—	
66-10	Chalk Creek Debris Basin	Chalk Creek	Fillmore, Utah	60.8	*60	July	1936	.5	44.12	.002	*70	[6].73	1,112.9	SCS
	do	do	do	—	—	—	1946	10	0	0	*75	[7].14	228.7	
	do	do	do	—	—	Mar.	1955	8.5	0	0	75	—	—	
66-11a	Fiddlers Canyon Debris Basin	Fiddlers Canyon	Cedar City, Utah	12.6	12.6	—	1947	—	—	—	—	—	—	SCS
	do	do	do	—	—	Nov.	1956	9	*15.00	*.089	*70	[8].145	221	
66-12a	Mill Canyon Retarding Structure	Mill Canyon	Glenwood, Utah	19.5	19.5	Dec.	1957	—	208	*1.600	—	—	-	SCS
	do	do	do	—	—	Dec.	1960	3	186	*1.431	*70	.38	578	

GREAT BASIN (NORTHWESTERN PART IN CALIFORNIA, NEVADA AND OREGON)

67-

GREAT BASIN
Humboldt, Carson, and Truckee River Basins

68-1	Willow Creek	Willow Creek	Elko, Nev.	112	111	—	1924	—	15,300	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1939	15	15,090	—	—	.15	—	
68-2	Elko Summit Reservoir No. 1	(ephemeral)	do	1.25	1.25	Mar.	1941	—	7.44	.223	—	—	—	SCS
	do	do	do	—	—	Aug.	1963	22.4	6.38	.192	66	.04	57.5	
68-3	Elko Summit Reservoir No. 2	do	do	.63	.63	Aug.	1945	—	5.45	.324	—	—	—	SCS
	do	do	do	—	—	Aug.	1963	18	4.96	.295	66.9	.05	73	
68-4	Dorsey Creek Reservoir	Dorsey Creek	do	9.16	9.14	Oct.	1952	—	131	.358	—	—	—	SCS
	do	do	do	—	—	Sept.	1963	10.9	128	.350	56.9	[8].12	[8]149	

GREAT BASIN
Owens, Walker, and Mono Lake Drainages

69-1	Weber	Walker River	Shurz, Nev.	2,000	2,440	Sept.	1935	—	13,200	—	—	—	—	SCS
	do	do	do	—*	—	Sept.	1939	4	12,980	—	—	.02	—	

SALTON SEA AND SOUTHERN CALIFORNIA COASTAL AND GREAT BASIN DRAINAGE

70-1a	Fullerton Flood-Control Basin	Fullerton Creek	Brea, Calif.	[9]5.05	[9]4.95	Oct.	1941	—	753.5	—	—	—	—	CE
	do	do	do	—	—	Nov.	1944	3.1	743.0	—	—	.69	—	
	do	do	do	—	—	Mar.	1962	17.3	706.3	—	—	.42	—	
70-2	Lake Hodges	San Dieguito River	Escondido, Calif.	303	301	Jan.	1919	—	36,601	—	—	—	—	SCS
	do	do	do	—	—	July	1935	16.5	34,779	—	*65	.365	517	
	do	do	do	—	—	July	1948	13.0	33,482	—	—	.332	470	
70-3	Railroad Canyon	San Jacinto River	Elsinore, Calif.	718	651	May	1928	—	12,218	—	—	—	—	SCS
	do	do	do	—	—	June	1939	11	12,034	—	*60	.03	39	
70-4	Lake Sherwood	Triunfo Creek	Hollywood, Calif.	16	15.7	—	1905	—	2,870	—	—	—	—	SCS
	do	do	do	—	—	Mar.	1936	31	2,792	—	*50	.16	174	
	do	do	do	—	—	Mar.	1938	2	2,772	—	—	.44	697	
70-5	Stone Canyon	Stone Canyon Creek	Sawtelle, Calif.	1.36	1.16	May	1921	—	7,998	—	—	—	—	SCS
	do	do	do	—	—	June	1939	18.1	7,949	—	*60	2.34	3,058	
70-6	Bonita Canyon	Bonita Creek	Orange, Calif.	4.00	4.00	Jan.	1938	—	326	—	—	—	—	SCS
	do	do	do	—	—	June	1939	[10]2	305	—	*60	2.63	3,437	

[1] Dam was raised about 6 ft. in 1952.
[2] Includes above crest deposits.
[3] Formed by original rock-filled dam 58 feet high, which was subsequently raised in 1900, 1902, and 1916.
[4] Computed from Size analysis.
[5] Original survey made in 1908.
[6] Basin filled with sediment by one summer flash flood.
[7] Has no meaning as a large portion of the sediment passed through the basin.
[8] Includes some deposition above crest of spillway.
[9] Drainage area and sediment contributing area were increased when Loftus diversion channel was completed on Dec. 21, 1956.
[10] Based on runoff seasons.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
						SALTON SEA AND SOUTHERN CALIFORNIA COASTAL AND GREAT BASIN DRAINAGE (Continued)							
70-7	Bouquet Canyon	Bouquet Creek	San Fernando, Calif.	12.6	11.6	Mar. 1934	—	36,500	—	—	—	—	SCS
	do	do	do	—	—	June 1939	1/5	36,436	—	60	1.10	958	
70-8a	Chatsworth	Trib. of Los Angeles River	do	5.40	4.65	Apr. 1918	—	10,125	—	—	—	—	SCS
	do	do	do	—	—	June 1939	21	10,077	—	60	.51	444	
70-9	Lake Hemet	Trib. of San Jacinto River	Hemet, Calif.	66.0	65.3	— 1892	—	14,000	—	—	—	—	SCS
	do	do	do	—	—	June 1940	48	11,702	—	75	.73	1,198	
70-10	Laguna	Trib. of Newport Bay	Orange, Calif.	.75	.72	Feb. 1938	—	274	—	—	—	—	SCS
	do	do	do	—	—	June 1939	1/2	264	—	60	5.56	6,860	
70-11	Encino	Encino Creek	Los Angeles, Calif.	1.42	1.30	May 1921	—	3,229	—	—	—	—	SCS
	do	do	do	—	—	June 1939	18	3,210	—	60	.815	710	
70-12	Fairmont	Antelope Valley River	Lancaster, Calif.	2.64	2.37	Oct. 1913	—	7,487	—	—	—	—	SCS
	do	do	do	—	—	June 1939	1/26	7,393	—	—	1.53	—	
70-13b	Little Rock Irrigation Dist.	Little Rock Creek	Palmdale, Calif.	68.0	67.84	Apr. 1924	—	4,587	—	—	—	—	SCS
	do	do	do	—	—	Jan. 1936	1/11.0	4,139	—	85	.10	185	
	do	do	do	—	—	June 1938	3.0	3,668	—	—	2.41	6,460	
	do	do	do	—	—	Oct. 1943	5.0	3,606	—	—	.72	1,330	
	do	do	do	—	—	Dec. 1946	3.0	3,352	—	—	.25	460	
	do	do	do	—	—	Oct. 1951	5.0	3,297	—	85	.15	278	
	do	do	do	—	—	Aug. 1953	2.2	3,313	—	85	3/—	3/—	
70-14a	Live Oak Dam	Live Oak Creek	LaVerne, Calif.	2.3	2.3	— 1919 3/	—	247	—	—	—	—	CE
	do	do	do	—	—	Mar. 1929	6.3	247	—	—	0	—	
	do	do	do	—	—	Mar. 1936	7.0	242	—	—	.31	—	
	do	do	do	—	—	May 1938	2.1	238	—	—	2.90	—	
	do	do	do	—	—	Nov. 1952	14.5	221	—	—	.21	—	
	do	do	do	—	—	Dec. 1961	9.0	170	—	—	2.46	—	
	do	do	do	—	—	Dec. 1962	1.0	166	—	—	1.76	—	
	do	do	do	—	—	Mar. 1967	4.3	4/221.8	1.33	—	2.03	—	
	do	do	do	—	—	Jan. 1969	1.75	197.1	1.11	—	6.13	—	
	do	do	do	—	—	Oct. 1970	1.75	251.1	1.13	—	5.70	—	
70-15	Morena	Cottonwood Creek	San Diego, Calif.	112.0	109.4	Mar. 1910	—	66,767	—	—	—	—	SCS
	do	do	do	—	—	Dec. 1935	25.7	60,699	—	60	2.16	2,823	
	do	do	do	—	—	July 1948	12.6	58,933	—	—	1.28	1,673	
70-16b	Prado Flood-Control Reservoir	Santa Ana River	Corona, Calif.	2,233	1,131	Sept. 1941	—	222,840	—	—	—	—	CE
	do	do	do	—	—	Aug. 1960	18.9	216,960	—	—	.275	—	
70-17	Mockingbird Canyon 5/	Mockingbird Canyon	Arlington, Calif.	11.6	11.5	— 1914	—	6/1,000	—	—	—	—	SCS
	do	do	do	—	—	Mar. 1960	46	961	—	60	.130	170	
70-18a	Hansen Flood-Control Basin	Tujunga Creek	San Fernando, Calif.	147	146	Sept. 1940	—	35,600	2.06	—	—	—	CE
	do	do	do	—	—	July 1941	.8	35,200	2.02	—	4.10	—	
	do	do	do	—	—	Oct. 1943	2.3	34,100	1.96	—	3.78	—	
	do	do	do	—	—	Nov. 1945	2.1	33,900	1.92	—	2.06	—	
	do	do	do	—	—	Jan. 1962	16.2	33,265	1.91	—	.10	—	
	do	do	do	—	—	Aug. 1969	7.58	29,700	1.71	—	3.22	—	
70-19	Brea F. C. Basin	Brea Creek	Fullerton, Calif.	23.4	23.1	Mar. 1962	—	4,168	3.97	—	—	—	CE
	do	do	do	—	—	Oct. 1969	7.5	4,097	3.90	—	.41	—	
70-20c	Cogswell (San Gabriel Dam #2)	San Gabriel River	Azusa, Calif.	39.2	39.0	Apr. 1935	—	12,681	—	—	—	—	CE
	do	do	do	—	—	Jan. 1936	.8	12,298	—	—	—	—	
	do	do	do	—	—	Apr. 1938	2.2	10,786	—	—	17.6	—	
	do	do	do	—	—	Nov. 1939	1.6	11,029	—	—	3/—	—	
	do	do	do	—	—	Nov. 1940	1.0	11,102	—	—	3/—	—	
	do	do	do	—	—	Nov. 1941	1.0	10,915	—	—	4.79	—	
	do	do	do	—	—	Oct. 1943	1.9	10,903	—	—	5.59	—	
	do	do	do	—	—	Jan. 1945	1.2	10,536	—	—	3/—	—	
	do	do	do	—	—	Sept. 1946	1.7	10,597	—	—	3/—	—	
	do	do	do	—	—	Sept. 1947	1.0	10,636	—	—	3/—	—	
	do	do	do	—	—	Dec. 1957	10.2	10,585	—	—	.12	—	
	do	do	do	—	—	Oct. 1958	.7	10,666	.612	—	2.48	—	
	do	do	do	—	—	Nov. 1962	4.1	10,288	.643	—	1.6	—	
	do	do	do	—	—	June 1966	3.7	9,999	.662	—	1.62	—	
	do	do	do	—	—	Mar. 1969	2.75	9,339	.512	—	6.15	—	
70-21d	San Gabriel Dam No. 1	do	do	203	7/203	— 1937	—	8/53,344	—	—	—	—	CE
	do	do	do	—	—	Oct. 1938	.9	67,191	—	—	28.3	—	
	do	do	do	—	—	Nov. 1940	2.1	66,335	—	—	1.68	—	
	do	do	do	—	—	Sept. 1941	.8	65,862	—	—	—	—	
	do	do	do	—	—	Oct. 1942	1.0	65,759	—	—	.43	—	
	do	do	do	—	—	Sept. 1943	1.0	64,032	—	—	7.13	—	
	do	do	do	—	—	Oct. 1944	1.1	64,388	—	—	—	—	
	do	do	do	—	—	Nov. 1945	1.0	64,362	—	—	.19	—	
	do	do	do	—	—	Nov. 1948	3.0	63,825	—	—	.97	—	
	do	do	do	—	—	Nov. 1951	3.0	63,908	—	—	.23	—	
	do	do	do	—	—	Jan. 1953	1.2	63,853	—	—	.96	—	

	do	do	do	--	--	May	1954	1.3	44,013	--	--	.127	--	
	do	do	do	--	--	Aug.	1958	4.3	44,614	--	--	.193	--	
	do	do	do	--	--	Sept.	1961	3.1	44,366	--	--	.329	--	
	do	do	do	--	--	Nov.	1962	1.2	43,642	.51	--	2.49	--	
	do	do	do	--	--	Dec.	1965	3.1	42,371	.49	--	2.02	--	
	do	do	do	--	--	Apr.	1966	.3	42,075	.51	--	--	--	
	do	do	do	--	--	Aug.	1967	1.3	40,697	.46	--	5.22	--	
	do	do	do	--	--	Feb.	1969	1.5	38,778	.45	--	8.86	--	
	do	do	do	--	--	May	1969	.25	36,748	.42	--	--	--	
	do	do	do	--	--	Oct.	1969	.42	37,822	.43	--	--	--	
	do	do	do	--	--	Oct.	1970	1.0	41,549	.48	--	--	--	
70-22b	Morris Reservoir	do	do	211.3	9/210.7	Jan.	1934	--	39,316	--	--	--	--	CE
	do	do	do	--	--	Jan.	1936	2.1	38,692	.45	--	1.41	--	
	do	do	do	--	--	Feb.	1938	2.1	37,175	.43	--	3.43	--	
	do	do	do	--	--	Oct.	1938	.7	36,655	.43	--	3.53	--	
	do	do	do	--	--	Dec.	1949	11.2	35,174	.41	--	.63	--	
	do	do	do	--	--	Sept.	1956	6.8	34,861	.41	--	.22	--	
	do	do	do	--	--	Nov.	1960	4.2	33,827	.40	--	1.17	--	
	do	do	do	--	--	Jan.	1967	6.2	32,961	.39	--	.66	--	
	do	do	do	--	--	Oct.	1970	3.8	30,085	.35	--	3.59	--	
70-23b	San Dimas F. C. Basin	San Dimas Wash	San Dimas, Calif.	16.2	16.1	Sept.	1922	--	4/1,496	--	--	--	--	CE
	do	do	do	--	--	Dec.	1935	13.2	1,373	--	--	.58	--	
	do	do	do	--	--	Oct.	1938	2.9	1,155	--	--	4.67	--	
	do	do	do	--	--	Nov.	1939	1.0	1,189	--	--	--	--	
	do	do	do	--	--	Dec.	1941	2.1	1,145	--	--	1.30	--	
	do	do	do	--	--	Oct.	1943	1.9	1,071	--	--	2.42	--	
	do	do	do	--	--	Nov.	1944	1.0	1,042	--	--	1.80	--	
	do	do	do	--	--	Oct.	1954	9.8	1,025	--	--	.11	--	
	do	do	do	--	--	Nov.	1961	7.1	705	.25	--	2.80	--	
	do	do	do	--	--	Apr.	1962	.4	729	.28	--	--	--	
	do	do	do	--	--	Aug.	1966	4.3	1,234	.49	--	16.0	--	
	do	do	do	--	--	Apr.	1967	.7	1,129	.25	--	--	--	
	do	do	do	--	--	Jan.	1969	1.75	847	.32	--	10.0	--	
	do	do	do	--	--	Feb.	1969	.08	756	.29	--	--	--	
	do	do	do	--	--	Nov.	1970	1.75	1,515	.47	--	1.12	--	
70-24c	Puddingstone Diversion Dam	do	do	19.9	10/19.7	Oct.	1929	--	4/148	.06	--	--	--	CE
	do	do	do	--	--	Jan.	1936	6.25	121	.05	--	1.66	--	
	do	do	do	--	--	Mar.	1938	2.17	50	.02	--	12.58	--	
	do	do	do	--	--	Nov.	1939	1.66	76	.03	--	--	--	
	do	do	do	--	--	Oct.	1942	2.92	101	.04	--	--	--	
	do	do	do	--	--	Sept.	1944	1.92	112	.05	--	--	--	
	do	do	do	--	--	Jan.	1952	7.33	119	.05	--	--	--	
	do	do	do	--	--	Sept.	1953	1.67	138	.06	--	--	--	
	do	do	do	--	--	Sept.	1962	9.0	203	.09	--	.28	--	
	do	do	do	--	--	May	1966	3.67	125	.05	--	1.07	--	
	do	do	do	--	--	Oct.	1967	1.41	200	.08	--	2.03	--	
	do	do	do	--	--	Jan.	1969	1.25	74	.03	--	5.13	--	
	do	do	do	--	--	Mar.	1969	.17	42	.02	--	--	--	
	do	do	do	--	--	Nov.	1970	1.66	156.3	.07	--	1.69	--	
70-25a	Puddingstone Dam F. C. Basin	Puddingstone Creek	do	32.2	11/11.0	--	1915	--	17,398	--	--	--	--	CE
	do	do	do	--	--	--	1927	0	17,398	--	--	--	--	
	do	do	do	--	--	Jan.	1941	13	17,190	--	--	1.45	--	
70-26c	Big Dalton F. C. Basin	Big Dalton Creek	Glendora, Calif.	4.5	4.5	Jan.	1930	--	4/1,194	--	--	--	--	CE
	do	do	do	--	--	Sept.	1931	1.7	1,039	--	--	20.26	--	
	do	do	do	--	--	Jan.	1935	3.3	1,053	--	--	--	--	
	do	do	do	--	--	Mar.	1938	3.1	969	--	--	6.02	--	
	do	do	do	--	--	Sept.	1943	5.5	953	--	--	.65	--	
	do	do	do	--	--	Oct.	1944	1.0	952	--	--	.22	--	
	do	do	do	--	--	Oct.	1958	14.0	951	--	--	.02	--	
	do	do	do	--	--	Nov.	1961	3.1	880	1.25	--	5.08	--	
	do	do	do	--	--	Jan.	1962	.2	869	1.24	--	--	--	
	do	do	do	--	--	Aug.	1966	4.7	752	1.14	--	5.53	--	
	do	do	do	--	--	Apr.	1967	.7	692	1.05	--	--	--	
	do	do	do	--	--	Jan.	1969	1.75	515	.78	--	.225	--	
	do	do	do	--	--	Mar.	1969	.17	452	.68	--	--	--	

1/ Based on runoff seasons.
2/ Not computed because of scour during period.
3/ Normal operation began 1922.
4/ Sediment removed at various times.
5/ Equalizing reservoir on Gage Canal System obtaining water from the Santa Ana River.
6/ No record of quantities sluiced.
7/ Includes San Gabriel Dams drainage area 39.2 sq. mi. as sediment is sluiced into San Gabriel Dam 1 drainage area.
8/ Storage capacity is regained by sluicing at various times.
9/ Includes Cogswell and San Gabriel Dam #1. (Drainage area 203 sq. mi.) Sediment sluiced into this area.
10/ Includes San Dimas drainage area (16.2 sq. mi.). Sediment sluiced into this area.
11/ 21.1 sq. mi. controlled by San Dimas, Live Oak Dam, and Puddingstone Diversion Dams.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY		PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET							AC.-FT.	TONS	
				SALTON SEA AND SOUTHERN CALIFORNIA COASTAL AND GREAT BASIN DRAINAGE (Continued)										
70-27h	Big Santa Anita F. C. Basin	Santa Anita Creek	Arcadia, Calif.	10.8	10.8	—	1923	—	1/1,376	—	—	—	—	CE
	do	do	do	—	—	Feb.	1935 2/	7.9	1,068	—	—	3.61	—	
	do	do	do	—	—	Feb.	1936	1.0	1,014	—	—	5.00	—	
	do	do	do	—	—	July	1938	2.4	688	—	—	12.6	—	
	do	do	do	—	—	Feb.	1940	1.6	710	—	—	7.18	—	
	do	do	do	—	—	Feb.	1942	2.0	701	—	—	1.43	—	
	do	do	do	—	—	Mar.	1943	1.1	588	—	—	11.2	—	
	do	do	do	—	—	Sept.	1943	.5	604	—	—	—	—	
	do	do	do	—	—	May	1944	.7	697	—	—	—	—	
	do	do	do	—	—	Jan.	1947	2.7	728	—	—	.44	—	
	do	do	do	—	—	Dec.	1952	5.9	728	—	—	.22	—	
	do	do	do	—	—	July	1954	1.6	584	—	—	9.15	—	
	do	do	do	—	—	Feb.	1956	1.5	613	—	—	1.30	—	
	do	do	do	—	—	Sept.	1958	2.6	587	—	—	.94	—	
	do	do	do	—	—	Apr.	1962	3.6	630	.16	—	3.31	—	
	do	do	do	—	—	Sept.	1966	4.4	552	.14	—	1.90	—	
	do	do	do	—	—	Oct.	1968	2.1	600	.14	—	3.09	—	
	do	do	do	—	—	Mar.	1969	.42	298	.06	—	—	—	
	do	do	do	—	—	Nov.	1970	1.67	836	.16	—	.94	—	
70-28e	Big Tujunga F. C. Basin	Big Tujunga Creek	Sunland, Calif.	82.3	82.2	—	1931	—	1/6,240	—	—	—	—	CE
	do	do	do	—	—	May	1938	6.9	4,734	—	—	2.65	—	
	do	do	do	—	—	Oct.	1939	1.5	4,488	—	—	2.00	—	
	do	do	do	—	—	Feb.	1940	.3	4,568	—	—	2.64	—	
	do	do	do	—	—	July	1941	1.5	4,625	—	—	7.33	—	
	do	do	do	—	—	Feb.	1943	1.5	4,063	—	—	3.10	—	
	do	do	do	—	—	Apr.	1943	.2	4,236	.28	—	—	—	
	do	do	do	—	—	June	1944	1.1	4,235	.28	—	2.19	—	
	do	do	do	—	—	Oct.	1953	9.3	4,099	.27	—	.18	—	
	do	do	do	—	—	June	1958	4.8	4,123	.27	—	.28	—	
	do	do	do	—	—	July	1962	4.1	4,065	.26	—	.40	—	
	do	do	do	—	—	Oct.	1966	4.25	3,819	.25	—	.71	—	
	do	do	do	—	—	Feb.	1969	2.33	2,758	.18	—	6.03	—	
	do	do	do	—	—	Oct.	1970	1.67	6,027	.39	—	—	—	
70-29b	Devil's Gate F. C. Basin	Arroyo Seco River	Pasadena, Calif.	31.9	31.7	—	1920	—	1/4,601	—	—	—	—	CE
	do	do	do	—	—	June	1933	13.0	4,554	—	—	.11	—	
	do	do	do	—	—	Sept.	1934	1.2	4,127	—	—	11.15	—	
	do	do	do	—	—	Jan.	1935	.3	3,996	—	—	13.69	—	
	do	do	do	—	—	June	1938	3.5	2,967	—	—	9.22	—	
	do	do	do	—	—	Jan.	1942	3.6	2,728	—	—	2.08	—	
	do	do	do	—	—	Jan.	1943	1.0	2,906	.38	—	5.02	—	
	do	do	do	—	—	Fall	1948	5.7	2,561	.39	—	1.25	—	
	do	do	do	—	—	July	1952	3.8	2,630	.40	—	1.06	—	
	do	do	do	—	—	Sept.	1955	3.3	2,709	.41	—	.58	—	
	do	do	do	—	—	Dec.	1959	4.2	2,839	.43	—	.63	—	
	do	do	do	—	—	May	1961	1.4	2,886	.44	—	4.87	—	
	do	do	do	—	—	May	1962	1.0	2,769	.42	—	9.59	—	
	do	do	do	—	—	Sept.	1966	4.3	2,598	.50	—	4.20	—	
	do	do	do	—	—	Nov.	1969	3.2	2,022	.30	—	7.32	—	
70-30d	Eaton Wash (Eaton Wash Dam)	Eaton Creek	do	9.5	9.4	—	1936	—	1/956	—	—	—	—	CE
	do	do	do	—	—	Feb.	1937	1.0	965	—	—	1.17	—	
	do	do	do	—	—	May	1938	1.3	698	—	—	20.2	—	
	do	do	do	—	—	Dec.	1938	.6	699	—	—	—	—	
	do	do	do	—	—	Oct.	1940	1.8	711	—	—	—	—	
	do	do	do	—	—	Aug.	1941	.8	661	—	—	—	—	
	do	do	do	—	—	Sept.	1942	1.1	722	—	—	—	—	
	do	do	do	—	—	Oct.	1943	1.1	632	.29	—	12.7	—	
	do	do	do	—	—	Oct.	1944	1.0	607	.27	—	2.7	—	
	do	do	do	—	—	Oct.	1946	2.0	676	.31	—	0	—	
	do	do	do	—	—	June	1947	.7	661	.30	—	2.0	—	
	do	do	do	—	—	June	1950	3.0	665	.30	—	0	—	
	do	do	do	—	—	Jan.	1952	1.6	703	.32	—	0.1	—	
	do	do	do	—	—	May	1957	5.3	655	.30	—	1.0	—	
	do	do	do	—	—	Nov.	1958	1.5	725	.33	—	.90	—	
	do	do	do	—	—	Sept.	1961	2.9	807	.37	—	2.48	—	
	do	do	do	—	—	Oct.	1963	2.0	828	.37	—	.53	—	
	do	do	do	—	—	Feb.	1964	.2	872.8	.40	—	—	—	
	do	do	do	—	—	Apr.	1966	2.2	769.7	.35	—	4.98	—	
	do	do	do	—	—	July	1967	1.3	758.4	.36	—	6.49	—	
	do	do	do	—	—	Dec.	1969	2.4	879	.40	—	11.3	—	

70-31c	Pacoima F. C. Basin	Pacoima Canyon	San Fernando, Calif.	28.2	28.2	Oct. 1929	—	1/6,060	—	—	--	—	CE
	do	do	do	--	—	Jan. 1936	6.3	5,592	—	—	2.63	—	
	do	do	do	--	--	Mar. 1938	2.2	5,004	—	—	9.47	—	
	do	do	do	--	—	Oct. 1942	4.6	4,837	—	—	1.29	—	
	do	do	do	—	--	Dec. 1944	2.1	4,714	—	--	2.84	—	
	do	do	do	—	—	Oct. 1954	9.8	4,787	—	—	.15	—	
	do	do	do	—	—	June 1958	3.8	4,651	—	—	2.97	—	
	do	do	do	—	—	May 1962	3.9	4,581	.83	—	.64	—	
	do	do	do	--	—	Aug. 1966	4.25	4,453	.81	—	1.08	—	
	do	do	do	--	--	Mar. 1969	2.58	3,841	.70	—	8.40	—	
70-32	Santiago	Santiago Creek	Villa Park, Calif.	[illegible]	62.1	Dec. 1931	—	25,000	1.619	—	—	--	CE
	do	do	do	--	--	Sept. 1968	16.8	24,840	1.609	—	.15	—	
70-33b	Sawpit F. C. Basin	Sawpit Creek	Monrovia, Calif.	3.3	3.3	— 1923 3/	—	1/476	—	--	--	—	CE
	do	do	do	--	--	Oct. 1935	8.3	389	—	—	3.18	—	
	do	do	do	—	—	May 1938	2.6	320	—	—	8.03	—	
	do	do	do	--	--	May 1941	3.0	342	--	—	2.94	—	
	do	do	do	--	—	Dec. 1943	2.6	322	—	—	3.03	—	
	do	do	do	--	—	Mar. 1954	10.2	305	—	—	.51	—	
	do	do	do	--	--	Sept. 1959	5.5	289	—	—	3.03	—	
	do	do	do	--	—	May 1962	2.7	271.8	.52	—	1.91	—	
	do	do	do	--	—	Aug. 1964	2.2	308.5	.61	—	5.09	—	
	do	do	do	--	--	July 1966	1.9	284.3	.55	—	3.82	—	
	do	do	do	--	--	May 1967	.7	269.2	.48	—	—	—	
	do	do	do	--	--	Nov. 1969	2.7	391.0	.47	—	16.39	—	
70-34b	Sierra Madre Debris Dam	Little Santa Anita Creek	Sierra Madre, Calif.	2.39	2.39	Feb. 1928	--	1/50.3	—	—	—	—	CE
	do	do	do	--	--	Feb. 1930	2.0	50.3	—	--	—	—	
	do	do	do	--	—	Feb. 1932	2.0	47.4	—	—	.61	—	
	do	do	do	--	—	Nov. 1939	7.8	34.4	—	—	.70	—	
	do	do	do	--	--	Jan. 1944	4.1	44.3	—	--	—	—	
	do	do	do	--	--	Oct. 1944	.8	42.4	—	--	1.00	—	
	do	do	do	--	--	Oct. 1956	12.0	40.6	—	—	1.95	—	
	do	do	do	—	--	Sept. 1969	13.0	—	—	—	4/2.10	—	
	do	do	do	—	—	Sept. 1970	1.0	--	—	—	1.53	—	
70-35b	Thompson Creek F. C. Basin	Thompson Creek	Claremont, Calif.	3.5	3.5	-- 1916	—	1/692	3.51	—	—	—	CE
	do	do	do	--	—	Oct. 1932	5/4.6	643	3.26	—	3.06	—	
	do	do	do	--	--	Jan. 1943	10.3	612	3.11	—	.86	—	
	do	do	do	--	--	Sept. 1954	11.7	571	2.90	—	1.00	—	
	do	do	do	--	--	Jan. 1957	3.3	566	2.87	—	.86	—	
	do	do	do	--	--	June 1957	.4	570	2.89	—	--	—	
	do	do	do	—	--	Dec. 1959	1.5	578	2.93	--	—	--	
	do	do	do	--	--	July 1962	2.6	554	2.81	—	2.64	—	
	do	do	do	--	—	Feb. 1967	4.6	510	2.59	—	2.73	--	
	do	do	do	--	—	Feb. 1969	2.0	448	2.27	—	8.93	—	
70-36e	Wilbur Ave. Debris Basin	Aliso Creek	Northridge, Calif.	8.63	8.63	June 1962	--	1/--	—	—	4/—	—	CE
	do	do	do	--	--	Sept. 1963	1.33	--	—	—	2.37	—	
	do	do	do	—	--	Sept. 1964	1	—	—	—	2.72	—	
	do	do	do	—	—	Sept. 1967	3.0	—	—	—	.85	--	
	do	do	do	—	—	Sept. 1950	3.0	—	—	--	.25	—	
	do	do	do	—	—	Sept. 1952	2	—	—	—	2.09	—	
	do	do	do	--	--	Sept. 1957	5	—	—	--	.47	--	
	do	do	do	--	--	Sept. 1960	3	—	—	--	.90	--	
	do	do	do	--	—	Sept. 1962	2	--	—	—	1.29	—	
	do	do	do	--	—	Sept. 1965	3	--	—	—	.41	—	
	do	do	do	--	--	Sept. 1966	1	—	—	—	3.99	—	
	do	do	do	--	--	Sept. 1967	1	—	—	—	2.35	—	
	do	do	do	—	--	Sept. 1968	1	—	—	—	2.17	—	
	do	do	do	--	--	Sept. 1969	1	—	--	—	2.72	--	
70-37b	Altadena Debris Basin	Rubio Wash	Altadena, Calif.	.65	.65	Sept. 1947	1	1/--	—	—	4/1.34	—	CE
	do	do	do	--	--	Sept. 1948	1	--	—	--	1.92	--	
	do	do	do	--	--	Sept. 1949	1	—	—	—	1.35	--	
	do	do	do	--	—	Sept. 1950	1	—	—	—	1.58	--	
	do	do	do	--	--	Mar. 1952	1.5	—	—	--	1.75	--	
	do	do	do	--	—	Feb. 1956	3.9	--	—	—	1.11	—	
	do	do	do	--	--	Sept. 1957	1.6	--	—	—	1.23	—	
	do	do	do	—	--	Apr. 1958	.6	--	—	--	8.97	--	
	do	do	do	—	--	Sept. 1959	1.4	—	—	--	2.57	--	

1/ Sediment is removed at various times by sluicing and/or excavation.
2/ Date normal operation began - Mar. 1927.
3/ Date normal operation began - June 1927.

4/ Sedimentation values as computed by Los Angeles County FCD are based on complete water year.
5/ Storage and surface area assumed same at time of initial normal operations, Mar. 1928 as in 1916 survey.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

SALTON SEA AND SOUTHERN CALIFORNIA COASTAL AND GREAT BASIN DRAINAGE (Continued)

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
70-38a	Bailey Debris Dam	Bailey Channel	Sierra Madre, Calif.	0.60	0.60	Sept. 1946	1	1/—	—	—	2/0.04	—	CE
	do	do	do	—	—	Sept. 1947	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1948	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1952	2	—	—	—	.18	—	
	do	do	do	—	—	Sept. 1957	4.8	—	—	—	2.88	—	
	do	do	do	—	—	Sept. 1962	5.0	—	—	—	6.83	—	
	do	do	do	—	—	Sept. 1968	6.0	—	—	—	.86	—	
	do	do	do	—	—	Sept. 1970	2.0	—	—	—	2.33	—	
70-39a	Brand Debris Basin	Brand Canyon Channel	La Canada, Calif.	1.03	1.03	Sept. 1940	5	1/—	—	—	2/.58	—	CE
	do	do	do	—	—	Sept. 1941	1	—	—	—	.63	—	
	do	do	do	—	—	Sept. 1942	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1943	1	—	—	—	1.86	—	
	do	do	do	—	—	Sept. 1944	1	—	—	—	.19	—	
	do	do	do	—	—	Sept. 1945	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1946	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1947	1	—	—	—	.09	—	
	do	do	do	—	—	Sept. 1948	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1952	2	—	—	—	1.99	—	
	do	do	do	—	—	Sept. 1964	12	—	—	—	1.17	—	
	do	do	do	—	—	Sept. 1965	1	—	—	—	27.86	—	
	do	do	do	—	—	Sept. 1970	5	—	—	—	2.87	—	
70-40a	Dunsmuir Debris Basin	Dunsmuir Canyon Channel	Tujunga, Calif.	.84	.84	Sept. 1936	1	1/—	—	—	2/.69	—	CE
	do	do	do	—	—	Sept. 1938	2	—	—	—	29.0	—	
	do	do	do	—	—	Sept. 1940	2	—	—	—	8.14	—	
	do	do	do	—	—	Sept. 1941	1	—	—	—	8.76	—	
	do	do	do	—	—	Sept. 1942	1	—	—	—	.27	—	
	do	do	do	—	—	Sept. 1943	1	—	—	—	10.3	—	
	do	do	do	—	—	Sept. 1944	1	—	—	—	2.87	—	
	do	do	do	—	—	Sept. 1945	1	—	—	—	.56	—	
	do	do	do	—	—	Sept. 1946	1	—	—	—	3.13	—	
	do	do	do	—	—	Sept. 1947	1	—	—	—	1.15	—	
	do	do	do	—	—	Sept. 1948	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1952	2	—	—	—	4.07	—	
	do	do	do	—	—	Sept. 1954	2	—	—	—	.36	—	
	do	do	do	—	—	Sept. 1959	5	—	—	—	3.17	—	
	do	do	do	—	—	Sept. 1970	11	—	—	—	1.98	—	
70-41a	Eagle Debris Basin	Eagle Canyon	La Crescenta, Calif.	.61	.61	Sept. 1938	2	1/—	—	—	2/21.2	—	CE
	do	do	do	—	—	Sept. 1939	1	—	—	—	11.7	—	
	do	do	do	—	—	Sept. 1941	2	—	—	—	7.05	—	
	do	do	do	—	—	Sept. 1942	1	—	—	—	.34	—	
	do	do	do	—	—	Sept. 1943	1	—	—	—	15.7	—	
	do	do	do	—	—	Sept. 1944	1	—	—	—	4.99	—	
	do	do	do	—	—	Sept. 1945	1	—	—	—	1.11	—	
	do	do	do	—	—	Sept. 1946	1	—	—	—	.79	—	
	do	do	do	—	—	Sept. 1947	1	—	—	—	.41	—	
	do	do	do	—	—	Sept. 1948	1	—	—	—	.07	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1952	2	—	—	—	2.66	—	
	do	do	do	—	—	Aug. 1956	3.9	—	—	—	.02	—	
	do	do	do	—	—	Sept. 1960	4.1	—	—	—	1.67	—	
	do	do	do	—	—	Sept. 1961	1	—	—	—	2.05	—	
	do	do	do	—	—	Sept. 1966	5	—	—	—	5.28	—	
	do	do	do	—	—	Sept. 1970	4	—	—	—	5.07	—	
70-42a	Fair Oaks Debris Basin	Fair Oaks Canyon	Altadena, Calif.	.21	.21	Sept. 1936	1	1/—	—	—	2/46.4	—	CE
	do	do	do	—	—	Sept. 1938	2	—	—	—	47.6	—	
	do	do	do	—	—	Sept. 1941	3	—	—	—	3.9	—	
	do	do	do	—	—	Sept. 1943	2	—	—	—	4.8	—	
	do	do	do	—	—	Sept. 1944	1	—	—	—	.24	—	
	do	do	do	—	—	Sept. 1945	1	—	—	—	1.71	—	
	do	do	do	—	—	Sept. 1946	1	—	—	—	2.06	—	
	do	do	do	—	—	Sept. 1947	1	—	—	—	1.95	—	

	do	do	do	—	—	Sept. 1948	1	—	—	—	.019	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1952	2	—	—	—	4.57	—	
	do	do	do	—	—	Oct. 1956	4.1	—	—	—	.16	—	
	do	do	do	—	—	June 1958	1.7	—	—	—	19.05	—	
	do	do	do	—	—	Sept. 1963	5.2	—	—	—	4.76	—	
	do	do	do	—	—	Sept. 1966	3	—	—	—	10.16	—	
	do	do	do	—	—	Sept. 1968	2	—	—	—	3.10	—	
	do	do	do	—	—	Sept. 1969	1	—	—	—	46.9	—	
70-43b	Fern Debris Basin	Fern Canyon	do	.30	.30	Sept. 1936	1	1/—	—	—	2/31.2	—	CE
	do	do	do	—	—	Sept. 1938	2	—	—	—	41.3	—	
	do	do	do	—	—	Sept. 1941	3	—	—	—	3.30	—	
	do	do	do	—	—	Sept. 1942	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1943	1	—	—	—	22.0	—	
	do	do	do	—	—	Sept. 1944	1	—	—	—	6.17	—	
	do	do	do	—	—	Sept. 1945	1	—	—	—	3.07	—	
	do	do	do	—	—	Sept. 1946	1	—	—	—	2.77	—	
	do	do	do	—	—	Sept. 1947	1	—	—	—	.43	—	
	do	do	do	—	—	Sept. 1948	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1952	2	—	—	—	5.47	—	
	do	do	do	—	—	Sept. 1964	12	—	—	—	1.23	—	
	do	do	do	—	—	Sept. 1966	2	—	—	—	16.67	—	
	do	do	do	—	—	Sept. 1968	2	—	—	—	11.5	—	
	do	do	do	—	—	Sept. 1969	1	—	—	—	24.6	—	
70-44c	Gould Debris Basin	Gould Canyon Channel	La Canada, Calif.	.47	.47	Sept. 1948	1	1/—	—	—	2/0	—	CE
	do	do	do	—	—	Sept. 1950	2	—	—	—	0	—	
	do	do	do	—	—	Sept. 1952	2	—	—	—	5.32	—	
	do	do	do	—	—	June 1958	5.8	—	—	—	3.29	—	
	do	do	do	—	—	Sept. 1962	4.2	—	—	—	5.83	—	
	do	do	do	—	—	Sept. 1966	4.0	—	—	—	9.04	—	
	do	do	do	—	—	Sept. 1968	2	—	—	—	1.98	—	
	do	do	do	—	—	Sept. 1969	1	—	—	—	28.7	—	
70-45a	Haines Debris Basin	Haines Canyon	Tujunga, Calif.	1.53	1.53	Sept. 1938	3	1/—	—	—	2/6.93	—	CE
	do	do	do	—	—	Sept. 1940	2	—	—	—	2.31	—	
	do	do	do	—	—	Sept. 1941	1	—	—	—	5.09	—	
	do	do	do	—	—	Sept. 1943	2	—	—	—	4.22	—	
	do	do	do	—	—	Sept. 1944	1	—	—	—	3.58	—	
	do	do	do	—	—	Sept. 1945	1	—	—	—	2.99	—	
	do	do	do	—	—	Sept. 1946	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1947	1	—	—	—	.37	—	
	do	do	do	—	—	Sept. 1948	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	0	—	
	do	do	do	—	—	May 1952	1.6	—	—	—	1.57	—	
70-46a	Hall's Debris Basin	Hall - Beckley Canyon	La Canada, Calif.	3/.84	.84	Sept. 1936	1	1/—	—	—	2/17.2	—	CE
	do	do	do	—	—	Sept. 1937	1	—	—	—	13.7	—	
	do	do	do	—	—	Sept. 1938	1	—	—	—	75.4	—	
	do	do	do	—	—	Sept. 1941	3	—	—	—	12.0	—	
	do	do	do	—	—	Sept. 1943	2	—	—	—	17.9	—	
	do	do	do	—	—	Sept. 1944	1	—	—	—	6.10	—	
	do	do	do	3/1.06	1.06	Sept. 1945	1	—	—	—	2.97	—	
	do	do	do	—	—	Sept. 1946	1	—	—	—	1.00	—	
	do	do	do	—	—	Sept. 1947	1	—	—	—	2.60	—	
	do	do	do	—	—	Sept. 1948	1	—	—	—	—	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	—	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	—	—	
	do	do	do	—	—	Sept. 1952	2	—	—	—	6.40	—	
	do	do	do	—	—	Feb. 1957	4.4	—	—	—	.14	—	
	do	do	do	—	—	Sept. 1959	2.6	—	—	—	11.03	—	
	do	do	do	—	—	Sept. 1962	3	—	—	—	.063	—	
	do	do	do	—	—	Sept. 1964	2	—	—	—	4.48	—	
	do	do	do	—	—	Sept. 1968	4	—	—	—	3.49	—	
	do	do	do	—	—	Sept. 1969	1	—	—	—	34.4	—	

1/ Capacity of debris basin varies. Debris excavated at various times.
2/ Sedimentation values as computed by LACFCD are based on complete water year.

3/ Drainage area 0.84 sq. mi. to 1945; 1.0659 sq. mi. beginning 1945.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
						SALTON SEA AND SOUTHERN CALIFORNIA COASTAL AND GREAT BASIN DRAINAGE (Continued)							
70-47c	Hay Debris Basin	Hay Canyon Channel	La Canada, Calif.	0.20	0.20	Sept. 1938	2	1/—	—	—	2/20.3	—	CE
	do	do	do	—	—	Sept. 1939	1	—	—	—	6.30	—	
	do	do	do	—	—	Sept. 1940	1	—	—	—	.55	—	
	do	do	do	—	—	Sept. 1941	1	—	—	—	1.90	—	
	do	do	do	—	—	Sept. 1943	2	—	—	—	4.75	—	
	do	do	do	—	—	Sept. 1944	1	—	—	—	.65	—	
	do	do	do	—	—	Sept. 1945	1	—	—	—	1.55	—	
	do	do	do	—	—	Sept. 1946	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1947	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1948	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1952	2	—	—	—	2.30	—	
	do	do	do	—	—	Sept. 1961	9	—	—	—	3.58	—	
	do	do	do	—	—	Sept. 1964	3	—	—	—	5.95	—	
	do	do	do	—	—	Sept. 1968	4	—	—	—	3.25	—	
	do	do	do	—	—	Sept. 1970	2	—	—	—	11.5	—	
70-48c	Las Flores Debris Basin	Las Flores Channel	Altadena, Calif.	.45	.45	Sept. 1936	1	1/—	—	—	2/14.7	—	CE
	do	do	do	—	—	Sept. 1938	2	—	—	—	38.2	—	
	do	do	do	—	—	Sept. 1939	1	—	—	—	1.53	—	
	do	do	do	—	—	Sept. 1943	4	—	—	—	4.40	—	
	do	do	do	—	—	Sept. 1944	1	—	—	—	3.71	—	
	do	do	do	—	—	Sept. 1945	1	—	—	—	.93	—	
	do	do	do	—	—	Sept. 1946	1	—	—	—	.96	—	
	do	do	do	—	—	Sept. 1947	1	—	—	—	.67	—	
	do	do	do	—	—	Sept. 1948	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1952	2	—	—	—	1.39	—	
	do	do	do	—	—	Sept. 1963	11	—	—	—	4.00	—	
	do	do	do	—	—	Sept. 1966	3	—	—	—	8.82	—	
	do	do	do	—	—	Sept. 1969	3	—	—	—	13.0	—	
70-49d	Lincoln Debris Basin	Lincoln Canyon	do	.50	.50	Sept. 1936	1	1/—	—	—	2/9.86	—	CE
	do	do	do	—	—	Sept. 1938	2	—	—	—	17.8	—	
	do	do	do	—	—	Sept. 1940	2	—	—	—	.92	—	
	do	do	do	—	—	Sept. 1941	1	—	—	—	15.1	—	
	do	do	do	—	—	Sept. 1943	2	—	—	—	6.48	—	
	do	do	do	—	—	Sept. 1944	1	—	—	—	2.32	—	
	do	do	do	—	—	Sept. 1945	1	—	—	—	.58	—	
	do	do	do	—	—	Sept. 1946	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1947	1	—	—	—	2.00	—	
	do	do	do	—	—	Sept. 1948	1	—	—	—	—	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	—	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	—	—	
	do	do	do	—	—	Mar. 1952	1.4	—	—	—	1.82	—	
	do	do	do	—	—	Sept. 1961	9.5	—	—	—	.10	—	
	do	do	do	—	—	Sept. 1967	6	—	—	—	3.34	—	
	do	do	do	—	—	Sept. 1969	2	—	—	—	16.20	—	
70-50c	Nichols Debris Basin	Nichols Canyon	Hollywood, Calif.	.96	.96	Sept. 1942	5	1/—	—	—	2/3.10	—	CE
	do	do	do	—	—	Sept. 1943	1	—	—	—	1.98	—	
	do	do	do	—	—	Sept. 1944	1	—	—	—	.48	—	
	do	do	do	—	—	Sept. 1945	1	—	—	—	.20	—	
	do	do	do	—	—	Sept. 1946	1	—	—	—	.15	—	
	do	do	do	—	—	Sept. 1947	1	—	—	—	3.72	—	
	do	do	do	—	—	Sept. 1948	1	—	—	—	.29	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	.38	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	.80	—	
	do	do	do	—	—	Apr. 1952	1.6	—	—	—	5.82	—	
	do	do	do	—	—	Feb. 1956	3.8	—	—	—	.47	—	
	do	do	do	—	—	Sept. 1959	3.6	—	—	—	1.39	—	
	do	do	do	—	—	Sept. 1960	1	—	—	—	3.29	—	
	do	do	do	—	—	Sept. 1966	6	—	—	—	1.06	—	
	do	do	do	—	—	Sept. 1968	2	—	—	—	2.15	—	
	do	do	do	—	—	Sept. 1970	2	—	—	—	2.77	—	

70-51b	Paradise Debris Basin	Paradise Canyon	La Canada, Calif.	.96	.96	Sept. 1945	1	1/--	—	—	2/1.18	—	CE
	do	do	do	—	—	Sept. 1946	1	—	—	—	.92	—	
	do	do	do	—	—	Sept. 1947	1	—	—	—	1.00	—	
	do	do	do	—	—	Sept. 1948	1	—	—	—	.14	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	.10	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	0	—	
	do	do	do	.58	.58	Apr. 1952	1.4	—	—	—	5.38	—	
	do	do	do	—	—	Mar. 1956	3.9	—	—	—	.98	—	
70-52e	Pickens Debris Basin	Pickens Canyon	La Crescenta, Calif.	1.84	1.84	Sept. 1936	1	1/—	—	—	2/11.0	—	CE
	do	do	do	—	—	Sept. 1937	1	—	—	—	6.89	—	
	do	do	do	—	—	Sept. 1938	1	—	—	—	44.9	—	
	do	do	do	—	—	Sept. 1939	1	—	—	—	2.95	—	
	do	do	do	—	—	Sept. 1940	1	—	—	—	4.75	—	
	do	do	do	—	—	Sept. 1941	1	—	—	—	11.6	—	
	do	do	do	—	—	Sept. 1943	2	—	—	—	9.03	—	
	do	do	do	—	—	Sept. 1944	1	—	—	—	3.00	—	
	do	do	do	—	—	Sept. 1945	1	—	—	—	.51	—	
	do	do	do	—	—	Sept. 1946	1	—	—	—	.23	—	
	do	do	do	—	—	Sept. 1947	1	—	—	—	.37	—	
	do	do	do	—	—	Sept. 1948	1	—	—	—	.15	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1952	2	—	—	—	2.25	—	
	do	do	do	—	—	Sept. 1959	7	—	—	—	2.81	—	
	do	do	do	—	—	Sept. 1966	7	—	—	—	3.41	—	
	do	do	do	—	—	Sept. 1970	4	—	—	—	4.44	—	
70-53e	Rubio Debris Dam	Rubio Wash	Altadena, Calif.	1.3	1.3	Sept. 1946	3	1/—	—	—	2/.69	—	CE
	do	do	do	—	—	Sept. 1947	1	—	—	—	.32	—	
	do	do	do	—	—	Sept. 1948	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1952	2	—	—	—	1.23	—	
	do	do	do	—	—	Apr. 1957	4.7	—	—	—	.05	—	
	do	do	do	—	—	Sept. 1966	9.4	—	—	—	2.07	—	
	do	do	do	—	—	Sept. 1969	3	—	—	—	10.0	—	
70-54b	Scholl Debris Basin	Scholl Canyon	Glendale, Calif.	.66	.66	Sept. 1947	2	1/—	—	—	2/.32	—	CE
	do	do	do	—	—	Sept. 1948	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1952	2	—	—	—	1.45	—	
	do	do	do	—	—	Sept. 1961	9	—	—	—	.53	—	
	do	do	do	—	—	Sept. 1970	9	—	—	—	.88	—	
70-55e	Shields Debris Basin	Shields Channel	La Crescenta, Calif.	.27	.27	Sept. 1938	1	1/--	—	—	2/77.0	—	CE
	do	do	do	—	—	Sept. 1939	1	—	—	—	10.1	—	
	do	do	do	—	—	Sept. 1941	2	—	—	—	10.9	—	
	do	do	do	—	—	Sept. 1943	2	—	—	—	5.85	—	
	do	do	do	—	—	Sept. 1944	1	—	—	—	2.33	—	
	do	do	do	—	—	Sept. 1945	1	—	—	—	.52	—	
	do	do	do	—	—	Sept. 1946	1	—	—	—	1.04	—	
	do	do	do	—	—	Sept. 1947	1	—	—	—	.04	—	
	do	do	do	—	—	Sept. 1948	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1952	1.5	—	—	—	10.9	—	
	do	do	do	—	—	Sept. 1961	9.5	—	—	—	2.07	—	
	do	do	do	—	—	Sept. 1966	5	—	—	—	6.81	—	
70-56e	Snover Debris Basin	Snover Canyon	do	.23	.23	Sept. 1938	2	1/--	—	—	2/22.6	—	CE
	do	do	do	—	—	Sept. 1939	1	—	—	—	57.0	—	
	do	do	do	—	—	Sept. 1941	2	—	—	—	4.4	—	
	do	do	do	—	—	Sept. 1943	2	—	—	—	8.30	—	
	do	do	do	—	—	Sept. 1945	2	—	—	—	.65	—	
	do	do	do	—	—	Sept. 1952	7	—	—	—	1.09	—	
	do	do	do	—	—	Sept. 1961	9	—	—	—	.06	—	
	do	do	do	—	—	Sept. 1968	7	—	—	—	3.30	—	
	do	do	do	—	—	Sept. 1969	1	—	—	—	56.5	—	
70-57b	Sparr Debris Basin	Sparr Channel	Montrose, Calif.	.84	.84	Sept. 1947	.6	1/—	—	—	2/0	—	CE
	do	do	do	—	—	Sept. 1948	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	0	—	
	do	do	do	—	—	Mar. 1952	1.5	—	—	—	3.56	—	
	do	do	do	—	—	Feb. 1956	3.9	—	—	—	1.13	—	
70-58b	Stough Debris Basin	Stough Canyon Channel	Burbank, Calif.	1.65	1.65	Sept. 1943	3	1/—	—	—	2/4.22	—	CE
	do	do	do	—	—	Sept. 1944	1	—	—	—	2.84	—	
	do	do	do	—	—	Sept. 1945	1	—	—	—	1.76	—	
	do	do	do	—	—	Sept. 1946	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1947	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1950	3	—	—	—	0	—	
	do	do	do	—	—	Apr. 1952	1.6	—	—	—	2.73	—	
	do	do	do	—	—	Sept. 1958	6.4	—	—	—	.37	—	
	do	do	do	—	—	Sept. 1960	2.0	—	—	—	.75	—	

1/ Capacity of debris basin varies. Debris excavated at various times.

2/ Sedimentation values as computed by LACFCD are based on complete water year.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
SALTON SEA AND SOUTHERN CALIFORNIA COASTAL AND GREAT BASIN DRAINAGE (Continued)													
70-59e	Sunset Canyon Debris Dam (upper)	Sunset Canyon Channel	Burbank, Calif.	0.44	0.44	Sept. 1942	13	1/—	—	—	2/1.34	—	CE
	do	do	do	—	—	Sept. 1944	2	—	—	—	.84	—	
	do	do	do	—	—	Sept. 1945	1	—	—	—	.18	—	
	do	do	do	—	—	Sept. 1946	1	—	—	—	.20	—	
	do	do	do	—	—	Sept. 1950	4	—	—	—	0	—	
	do	do	do	—	—	Sept. 1952	2	—	—	—	2.41	—	
	do	do	do	—	—	Sept. 1959	7	—	—	—	1.68	—	
	do	do	do	—	—	Sept. 1966	7	—	—	—	7.80	—	
	do	do	do	—	—	Sept. 1969	3	—	—	—	6.20	—	
70-60	Vanalden Debris Basin	Vanalden Channel	Encino, Calif.	1.08	1.08	Sept. 1946	1	1/—	—	—	2/.32	—	CE
	do	do	do	—	—	Sept. 1947	1	—	—	—	.02	—	
	do	do	do	—	—	Sept. 1950	3	—	—	—	—	—	
70-61e	Verdugo Debris Basin	Verdugo Wash	Montrose, Calif.	15.5	10.0	Sept. 1938	3.58	1/—	—	—	2/1.83	—	CE
	do	do	do	—	—	Sept. 1941	3	—	—	—	.81	—	
	do	do	do	—	—	Sept. 1943	2	—	—	—	2.05	—	
	do	do	do	—	—	Sept. 1944	1	—	—	—	4.27	—	
	do	do	do	—	—	Sept. 1950	6	—	—	—	.06	—	
	do	do	do	—	—	Sept. 1952	2	—	—	—	.82	—	
	do	do	do	—	—	Sept. 1953	1	—	—	—	3.73	—	
	do	do	do	—	—	May 1956	2.67	—	—	—	.002	—	
	do	do	do	—	—	Jan. 1959	2.67	—	—	—	.89	—	
	do	do	do	—	—	Sept. 1960	1.66	—	—	—	.05	—	
	do	do	do	—	—	Sept. 1964	4	—	—	—	.55	—	
	do	do	do	—	—	Sept. 1966	2	—	—	—	1.71	—	
	do	do	do	—	—	Sept. 1968	2	—	—	—	.05	—	
	do	do	do	—	—	Sept. 1970	2	—	—	—	3.14	—	
70-62b	Ward Debris Basin (lower)	Ward Canyon	La Crescenta, Calif.	3/.64	3/.64	Sept. 1945	1	1/—	—	—	2/.51	—	CE
	do	do	do	—	—	Sept. 1946	1	—	—	—	1.63	—	
	do	do	do	—	—	Sept. 1947	1	—	—	—	4.06	—	
	do	do	do	—	—	Sept. 1948	1	—	—	—	1.00	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	.36	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	1.11	—	
	do	do	do	—	—	Feb. 1952	1.3	—	—	—	6.41	—	
	do	do	do	—	—	Apr. 1952	.2	—	—	—	—	—	
	do	do	do	—	—	Sept. 1956	4.4	—	—	—	.52	—	
	do	do	do	—	—	Sept. 1958	2.0	—	—	—	.70	—	
70-63b	West Ravine Debris Basin	West Ravine Canyon	Altadena, Calif.	.25	.25	Sept. 1936	1	1/—	—	—	2/47.85	—	CE
	do	do	do	—	—	Sept. 1937	1	—	—	—	45.42	—	
	do	do	do	—	—	Sept. 1938	1	—	—	—	73.85	—	
	do	do	do	—	—	Sept. 1940	2	—	—	—	3.60	—	
	do	do	do	—	—	Sept. 1941	1	—	—	—	21.4	—	
	do	do	do	—	—	Sept. 1942	1	—	—	—	.44	—	
	do	do	do	—	—	Sept. 1943	1	—	—	—	20.20	—	
	do	do	do	—	—	Sept. 1944	1	—	—	—	8.00	—	
	do	do	do	—	—	Sept. 1945	1	—	—	—	.80	—	
	do	do	do	—	—	Sept. 1946	1	—	—	—	1.56	—	
	do	do	do	—	—	Sept. 1947	1	—	—	—	1.52	—	
	do	do	do	—	—	Sept. 1948	1	—	—	—	.02	—	
	do	do	do	—	—	Sept. 1949	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1950	1	—	—	—	0	—	
	do	do	do	—	—	Sept. 1952	2	—	—	—	4.76	—	
	do	do	do	—	—	Sept. 1964	12	—	—	—	1.08	—	
	do	do	do	—	—	Sept. 1968	4	—	—	—	8.69	—	
	do	do	do	—	—	Sept. 1969	1	—	—	—	43.2	—	
70-64	Camp Marston	Bear Creek	Julian, Calif.	1.6	1.59	— 1918	—	44.1	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1951	33	36.9	—	60	.14	183.0	
70-65b	Cooks Canyon Debris Basin	Cooks Canyon	La Crescenta, Calif.	.58	.58	Mar. 1952	.2	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1961	9.5	—	—	—	2/.280	—	
	do	do	do	—	—	Sept. 1969	8	—	—	—	.028	—	
70-66	Auburn Debris Basin	Auburn River	Sierra Madre, Calif.	.19	.19	Dec. 1954	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1959	4.8	—	—	—	2.6	—	
70-67a	Bradbury Debris Basin	Bradbury Canyon	Monrovia, Calif.	.68	.68	Jan. 1955	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1959	4.67	—	—	—	2/8.15	—	
	do	do	do	—	—	Sept. 1961	2.0	—	—	—	1.62	—	
	do	do	do	—	—	Sept. 1966	5	—	—	—	6.65	—	
	do	do	do	—	—	Sept. 1969	3	—	—	—	23.5	—	

70-68a	Carter Debris Basin	Carter Canyon	Sierra Madre, Calif.	.12	.12	Dec. 1954	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1959	4.8	—	—	—	2/2.8	—	
	do	do	do	—	—	Sept. 1962	3	—	—	—	23.7	—	
	do	do	do	—	—	Sept. 1969	7	—	—	—	4.06	—	
70-69a	Bear Debris Basin	Bear Canyon	Glendale, Calif.	.59	.59	Jan. 1955	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1959	4.83	—	—	—	2/.55	—	
	do	do	do	—	—	Sept. 1964	5	—	—	—	2.10	—	
	do	do	do	—	—	Sept. 1965	1	—	—	—	24.74	—	
	do	do	do	—	—	Sept. 1966	1	—	—	—	21.19	—	
	do	do	do	—	—	Sept. 1969	3	—	—	—	19.0	—	
70-70	Floral (Upper) Debris Basin	Floral Canyon	Sierra Madre, Calif.	.06	.06	Mar. 1954	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1959	5.6	—	—	—	2.4	—	
70-71	La Tuna Debris Basin	La Tuna Canyon	Los Angeles, Calif.	5.34	5.34	Jan. 1956	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Feb. 1956	.1	—	—	—	—	—	
	do	do	do	—	—	July 1957	1.4	—	—	—	2/.04	—	
	do	do	do	—	—	Sept. 1958	1.2	—	—	—	.13	—	
	do	do	do	—	—	Sept. 1959	1.0	—	—	—	2.15	—	
	do	do	do	—	—	Sept. 1960	1.0	—	—	—	1.22	—	
70-72a	Lannan Debris Basin	Lannan Wash	Sierra Madre, Calif.	.25	.25	Mar. 1954	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1959	5.6	—	—	—	2/5.44	—	
	do	do	do	—	—	Sept. 1962	3	—	—	—	11.3	—	
	do	do	do	—	—	Sept. 1967	5	—	—	—	.48	—	
	do	do	do	—	—	Sept. 1970	3	—	—	—	20.4	—	
70-73b	Maddock Debris Basin	Maddock Canyon	Duarte, Calif.	.25	.25	Jan. 1955	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Mar. 1956	1.2	—	—	—	2/17.8	—	
	do	do	do	—	—	Sept. 1959	3.5	—	—	—	3.88	—	
	do	do	do	—	—	Sept. 1967	8.0	—	—	—	1.8	—	
	do	do	do	—	—	Sept. 1969	2	—	—	—	15.8	—	
	do	do	do	—	—	Sept. 1970	1	—	—	—	2.96	—	
70-74a	May No. 1 Debris Basin	May Canyon	San Fernando, Calif.	.70	.70	Aug. 1953	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1959	6.2	—	—	—	2/.78	—	
	do	do	do	—	—	Sept. 1967	8	—	—	—	7.20	—	
	do	do	do	—	—	Sept. 1968	1	—	—	—	23.3	—	
	do	do	do	—	—	Sept. 1969	1	—	—	—	104.0	—	
70-75a	May No. 2 Debris Basin	do	do	.09	.09	Aug. 1953	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1959	6.2	—	—	—	2/.36	—	
	do	do	do	—	—	Sept. 1967	8	—	—	—	11.9	—	
	do	do	do	—	—	Sept. 1968	1	—	—	—	45.5	—	
	do	do	do	—	—	Sept. 1969	1	—	—	—	28.9	—	
70-76a	McClure Debris Basin	McClure Canyon	Burbank, Calif.	.62	.62	Dec. 1953	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1959	5.8	—	—	—	2/6.6	—	
	do	do	do	—	—	Sept. 1961	2.0	—	—	—	9.27	—	
	do	do	do	—	—	Sept. 1965	4	—	—	—	2.9	—	
	do	do	do	—	—	Sept. 1966	1	—	—	—	10.0	—	
	do	do	do	—	—	Sept. 1967	1	—	—	—	9.52	—	
70-77	Rowley Debris Basin	Rowley Canyon	Tujunga, Calif.	.58	.58	Jan. 1954	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1959	5.67	—	—	—	2/.62	—	
	do	do	do	—	—	Sept. 1968	9	—	—	—	1.5	—	
	do	do	do	—	—	Sept. 1969	1	—	—	—	11.8	—	
70-78	Ruby - Upper Debris Basin	Ruby Canyon	Sierra Madre, Calif.	.21	.21	Jan. 1954	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1959	5.8	—	—	—	2/5.2	—	
	do	do	do	—	—	Sept. 1961	2.0	—	—	—	.29	—	
70-79a	Ruby - Lower Debris Basin	do	do	.10	.10	Dec. 1955	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1959	3.8	—	—	—	2/2.9	—	
	do	do	do	—	—	Sept. 1962	3	—	—	—	1.0	—	
	do	do	do	—	—	Sept. 1969	7	—	—	—	12.8	—	
70-80a	Sawpit Debris Dam	Sawpit Creek	Monrovia, Calif.	6.34	2.84	Jan. 1955	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1959	4.67	—	—	—	2/4.6	—	
	do	do	do	—	—	Sept. 1962	3	—	—	—	6.9	—	
	do	do	do	—	—	Sept. 1969	7	—	—	—	9.82	—	
70-81a	Sierra Madre Villa Debris Basin.	Sierra Madre Villa Canyon	Sierra Madre, Calif.	1.46	1.46	May 1958	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1959	1.4	—	—	—	2/.34	—	
	do	do	do	—	—	Sept. 1962	3	—	—	—	16.92	—	
	do	do	do	—	—	Sept. 1968	6	—	—	—	.18	—	
	do	do	do	—	—	Sept. 1970	2	—	—	—	45.89	—	
70-82a	Spinks Debris Basin	Spinks Canyon	Monrovia, Calif.	.44	.44	Dec. 1958	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1959	.8	—	—	—	2/—	—	
	do	do	do	—	—	Sept. 1962	3	—	—	—	1.66	—	
	do	do	do	—	—	Sept. 1969	7	—	—	—	5.32	—	
70-83a	Turnbull Debris Basin	Turnbull Canyon Wash	Whittier, Calif.	.99	.99	Jan. 1953	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1959	6.8	—	—	—	2/.28	—	
	do	do	do	—	—	Sept. 1968	9	—	—	—	1.33	—	
	do	do	do	—	—	Sept. 1969	1	—	—	—	16.0	—	
70-84a	Zachau Debris Basin	Zachau Canyon	Tujunga, Calif.	.35	.35	Aug. 1956	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1959	3.2	—	—	—	2/.80	—	
	do	do	do	—	—	Sept. 1961	2.0	—	—	—	.80	—	
	do	do	do	—	—	Sept. 1969	8	—	—	—	2.37	—	

1/ Capacity of debris basin varies. Debris excavated at various times.
2/ Sedimentation valves computed by Los Angeles County FCD are based on complete wateryear.
3/ Drainage area 0.35 sq. mi. through 1947; 0.64 after 1947.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
	SALTON SEA AND SOUTHERN CALIFORNIA COASTAL AND GREAT BASIN DRAINAGE (Continued)												
70-85a	Ward Debris Basin (Upper)	Ward Canyon	La Crescenta, Calif.	0.10	0.10	Nov. 1956	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1957	.8	—	—	—	2/30.1	—	
	do	do	do	—	—	Sept. 1958	1.0	—	—	—	39.3	—	
	do	do	do	—	—	Sept. 1959	1.0	—	—	—	17.5	—	
	do	do	do	—	—	Sept. 1967	8	—	—	—	.4	—	
	do	do	do	—	—	Sept. 1968	1	—	—	—	3.1	—	
	do	do	do	—	—	Sept. 1969	1	—	—	—	22.9	—	
70-86	Sepulveda Flood Control Basin	Los Angeles River	Van Nuys, Calif.	152	142	Nov. 1941	—	16,720	1.00	—	—	—	CE
	do	do	do	—	—	Nov. 1944	3.0	17,437	1.04	—	—	—	
	do	do	do	—	—	June 1961	16.6	17,296	1.03	—	.06	—	
70-87a	Santa Fe Flood Control Basin	San Gabriel River	Baldwin Park, Calif.	236.0	2/20.54	Apr. 1943	—	34,670	—	—	—	—	CE
	do	do	do	—	—	Nov. 1949	6.6	34,276	—	—	2.91	—	
	do	do	do	—	—	Mar. 1959	9.3	33,987	—	—	1.51	—	
	do	do	do	—	—	June 1961	2.2	33,385	—	—	13.32	—	
	do	do	do	—	—	Feb. 1967	5.7	32,716	—	—	5.72	—	
	do	do	do	—	—	Sept. 1968	1.6	1/34,916	—	—	—	—	
	do	do	do	—	—	Aug. 1969	.9	32,642	—	—	123.0	—	
70-88a	Bell IV (CSCA)	Bell Creek	Glendora, Calif.	.061	.061	Jan. 1966	—	2.30	.234	—	—	—	FS
	do	do	do	—	—	Dec. 1966	.96	—	—	—	1.16	—	
	do	do	do	—	—	Oct. 1968	1.86	—	—	—	0	—	
	do	do	do	—	—	Mar. 1969	.43	—	—	—	39.6	—	
	do	do	do	—	—	July 1970	1.41	—	—	—	.018	—	
70-89	Bell Debris Basin	do	Canoga Park, Calif.	7.0	7.0	— —	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1970	3	—	—	—	2/.71	—	
70-90	Carbon Canyon F. C. Basin	Carbon Canyon Creek	Brea, Calif.	19.3	19.3	Mar. 1961	—	7,033	10.0	—	—	—	CE
	do	do	do	—	—	Sept. 1969	8.5	6,615	9.45	—	2.54	—	
70-91	Childs Debris Basin	Childs Canyon	Glendale, Calif.	.31	.31	Sept. 1963	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1966	3	—	—	—	2/8.81	—	
70-92	Elmwood Debris Basin	Elmwood Canyon	Burbank, Calif.	.31	.31	— 1964	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1968	4	—	—	—	2/7.90	—	
	do	do	do	—	—	Sept. 1970	2	—	—	—	.10	—	
70-93	Eaglewild Debris Basin	Eaglewild Canyon	Glendora, Calif.	.4	.4	Sept. 1961	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1969	8	—	—	—	2/10.4	—	
	do	do	do	—	—	Sept. 1970	1	—	—	—	13.5	—	
70-94	Harrow Debris Basin	Harrow Canyon	Glendora, Calif.	.43	.43	Sept. 1958	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1969	11	—	—	—	2/6.77	—	
	do	do	do	—	—	Sept. 1970	1	—	—	—	16.3	—	
70-95	Rock East Debris Basin	Rock Canyon	Azusa, Calif.	.18	.18	Sept. 1967	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1969	2	—	—	—	2/46.7	—	
	do	do	do	—	—	Sept. 1970	1	—	—	—	2.78	—	
70-96	Kinneloa Debris Basin	Kinneloa Canyon	Pasadena, Calif.	.20	.20	Oct. 1964	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1966	1.92	—	—	—	2/10.30	—	
	do	do	do	—	—	Sept. 1969	3	—	—	—	21.65	—	
	do	do	do	—	—	Sept. 1970	1	—	—	—	12.70	—	
70-97	Kinneloa - West Debris Basin	do	Altadena, Calif.	.16	.16	Sept. 1966	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1969	3	—	—	—	2/30.8	—	
	do	do	do	—	—	Sept. 1970	1	—	—	—	19.4	—	
70-98	Little Dalton Debris Basin	Little Dalton	Glendora, Calif.	3.3	3.3	Feb. 1960	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1962	2.6	—	—	—	2/13.9	—	
	do	do	do	—	—	Sept. 1969	7	—	—	—	12.4	—	
70-99	Morgan Debris Basin	Morgan Canyon	do	.6	.6	Sept. 1964	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1969	5	—	—	—	2/2.95	—	
70-100	San Antonio F. C. Basin	San Antonio Creek	Claremont, Calif.	26.7	26.7	Aug. 1956	—	9,285	.90	—	—	—	CE
	do	do	do	—	—	July 1969	12.91	6,718	.65	—	7.65	—	
70-101	Schoolhouse Debris Basin	Mansfield Av. Storm Drn.	Olive View, Calif.	.28	.28	June 1963	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1967	4.2	—	—	—	2/11.1	—	
70-102	Sunset Debris Basin (Lower)	Sunset Canyon	Glendale, Calif.	.65	.65	Jan. 1965	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1967	2.7	—	—	—	2/5.58	—	
70-103	Santa Anita Debris Basin	Santa Anita Creek	Arcadia, Calif.	1.7	1.7	Dec. 1959	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1962	2.8	—	—	—	2/19.1	—	
	do	do	do	—	—	Sept. 1968	6	—	—	—	.67	—	
	do	do	do	—	—	Sept. 1970	2	—	—	—	50.0	—	
70-104	Wilson Debris Basin	Wilson Canyon	Olive View, Calif.	2.58	2.58	June 1963	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept. 1967	4.2	—	—	—	2/5.23	—	

70-105	Wildwood Debris Basin	Wildwood Creek	Newhall, Calif.	.65	.65	Sept.	1967	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept.	1969	2	—	—	—	2/9.85	—	
70-106	Winery Debris Basin	Winery Canyon	La Crescenta, Calif.	.18	.18	Sept.	1968	—	1/—	—	—	—	—	CE
	do	do	do	—	—	Sept.	1969	1	—	—	—	3/28.2	—	
SAN JOAQUIN AND KERN RIVER BASINS AND ADJACENT COASTAL DRAINAGE														
71-1	Upper Crystal Springs	Laguna Creek	San Francisco, Calif.	13.3	12.0	Jan.	1870	—	29,138	12.67	—	—	—	SCS
	do	do	do	—	—	Oct.	1935	57.8	28,199	12.24	—	1.41	—	
71-2	Atascadero Park Lake	Atascadero Creek	Atascadero, Calif.	1.0	1.0	—	1918	—	150	—	—	—	—	SCS
	do	do	do	—	—	Nov.	1947	29	136	—	*60	.48	627	
71-3	Haskins	N. Fk. Los Viboras Creek	Hollister, Calif.	4.04	4.01	—	1912	—	549	—	—	—	—	SCS
	do	do	do	—	—	May	1940	28	4/532	—	*60	4/.15	196	
71-4a	Salinas	Salinas River	Santa Margarita, Calif.	111	110	—	1942	—	26,000	—	—	—	—	SCS
	do	do	do	—	—	Nov.	1947	6	25,860	—	—	.21	—	
	do	do	do	--	—	Aug.	1953	6	25,590	—	40	.41	357	
71-5	Copperopolis	Penney Creek	Copperopolis, Calif.	2.06	2.01	—	1915	—	265	—	—	—	—	SCS
	do	do	do	—	--	Aug.	1945	30	263	—	55	.034	40.7	
71-6	Crane Valley	N. Fk. San Joaquin River	Madera, Calif.	54.5	52.7	—	1901	—	45,410	--	—	—	—	SCS
	do	do	do	—	—	June	1946	45	45,028	—	*62	.161	217	
71-7	Davis	Shaw Creek	Stockton, Calif.	7.87	7.62	—	1917	—	1,421	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1945	28	1,367	—	63	.251	344	
71-8	Don Pedro	Tuolumne River	Modesto, Calif.	1,001	996.5	Mar.	1923	—	289,000	—	—	—	—	SCS
	do	do	do	—	—	Nov.	1945	22.7	284,266	—	*62	.210	284	
71-9	Exchequer	Merced River	Merced, Calif.	1,027	1,022.4	Sept.	1926	—	289,000	—	—	—	—	SCS
	do	do	do	—	—	Mar.	1946	19.6	285,646	—	62	.167	226	
71-10	Gilmore	Trib. of Mormon Slough	Bellota, Calif.	5.02	4.92	Sept.	1917	—	579	—	—	—	—	SCS
	do	do	do	—	—	Aug.	1945	28	560	—	50	.132	144	
71-11	Mono	Ten Mile Creek	Fresno, Calif.	24.2	24.1	June	1909	—	1,410	—	—	—	—	SCS
	do	do	do	—	—	June	1946	37.0	1,383	—	*62	.030	40.5	
71-12	La Grange	Tuolumne River	Modesto, Calif.	1,501	5/1,501	Sept.	1895	—	2,332	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1905	10.1	1,068	—	*70	.083	127	
71-13	Lyons	S. Fk. Stanislaus River	Tuolumne, Calif.	6/40.0	39.7	June	1930	—	5,500	—	—	—	—	SCS
	do	do	do	—	—	June	1946	16	5,436	—	*70	.101	154	
71-14	McCarty	Trib. of Johnny Creek	San Andreas, Calif.	.35	.32	Dec.	1937	—	96.3	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1945	7.7	95.5	—	*45	.300	294	
71-15	Pardee	Mokelumne River	Lodi, Calif.	7/387	383.5	—	1929	—	210,000	—	—	—	—	SCS
	do	do	do	—	—	Aug.	1943	14	209,183	—	*62	.152	205	
71-16	Salt Springs Valley	Rock Creek	San Andreas, Calif.	20.3	18.4	—	1882	—	12,930	—	—	—	—	SCS
	do	do	do	—	—	July	1945	63	12,697	—	*50	.201	219	
71-17	(Upper) Bear River	Bear River	Placerville, Calif.	28.5	28.2	Sept.	1900	—	6,712	—	—	--	—	SCS
	do	do	do	—	—	June	1946	45.8	6,690	—	*70	.017	26	
71-18a	Woods Pond	Sycamore Creek	Danville, Calif.	.30	.30	—	1940	—	4.41	—	—	—	—	SCS
	do	do	do	—	--	—	1951	11	4.23	—	80	.053	92.3	
71-19	St. Mary's	Walnut Creek	Walnut Creek, Calif.	3.11	2.97	—	1928	—	134.01	—	—	—	—	SCS
	do	do	do	—	—	—	1951	23	88.99	—	65	.66	647	
71-20	Black Hills	Green Valley Creek	Danville, Calif.	.76	.76	—	1880	—	32.68	—	—	—	—	SCS
	do	do	do	—	—	—	1951	71	27.04	—	46	.11	110.2	
71-21	Lakewood	Walnut Creek	Concord, Calif.	.75	.74	—	1913	—	27.08	—	—	—	—	SCS
	do	do	do	—	—	—	1951	40	24.86	—	30	.076	49.66	
71-22	Contra Costa Country Club	Grayson Creek	Pacheco, Calif.	.20	.20	—	1934	—	5.75	—	—	—	—	SCS
	do	do	do	—	—	—	1951	17	5.24	—	40	.15	131	
71-23	Englehart Pond	Marsh Creek	Brentwood, Calif.	.033	.033	—	1945	—	1.34	—	—	—	—	SCS
	do	do	do	—	—	Jan.	1953	7	1.32	—	*60	.088	115	
71-24a	C. C. Anderson Pond	do	do	.083	.083	—	1945	—	1.920	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1952	7	1.864	—	*65	8/.096	8/136	
71-25	Upper Walter Keller Pond	do	do	.027	.027	—	1943	—	1.347	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1952	9	1.185	—	*40	8/.682	8/594	
71-26	F. J. Hoey Pond	Alhambra Creek	do	.065	.065	—	1947	—	2.45	—	—	—	—	SCS
	do	do	do	—	—	Jan.	1953	5	2.32	—	*55	8/.578	8/692.38	
71-27a	Lee Higgins Pond	Marsh Creek	do	.202	.199	—	1941	—	12.60	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1953	12	12.28	—	*40	.14	123	
71-28a	Souza	do	do	.319	.283	—	1946	—	176.49	—	—	—	—	SCS
	do	do	do	—	—	Jan.	1953	6	174.97	—	*30	.094	984	
71-29a	Ordway	do	do	.619	.613	—	1945	—	60.9	—	—	—	—	SCS
	do	do	do	—	—	Jan.	1953	7	60.07	—	*70	.19	290	
71-30	North Fork	Pacheco Creek	Hollister, Calif.	66.2	66.0	—	1938	—	5,803	—	—	—	—	SCS
	do	do	do	—	—	Apr.	1952	12.4	5,706	—	*45	8/.12	8/117.41	

1/ Capacity of debris basin varies. Debris excavated at various times.
2/ Sedimentation values as computed by Los Angeles County FCD are based on complete water year.
3/ Excludes drainage area above Morris Dam (215.46 sq. mi.).
4/ Some sediment sluiced out incidental to work on outlet.
5/ Before construction of Don Pedro Reservoir in 1923.
6/ Excluding drainage area of Strawberry Reservoir.
7/ Excluding noncontributing areas above Salt Springs and Bear River Reservoirs.
8/ Excluding pond area.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
				SAN JOAQUIN AND KERN RIVER BASINS AND ADJACENT COASTAL DRAINAGE (Continued)									
71-31a	Teakettle Reservoir No. 1	Teakettle Creek	Fresno, Calif.	0.77	0.77	Fall 1938	—	0.39	—	—	—	—	FS
	do	do	do	—	—	Fall 1948	10.0	.18	—	—	0.0271	—	
	do	do	do	—	—	Fall 1951	3.0	.002	—	—	.1100	—	
	do	do	do	—	—	Fall 1955	4.0	0	—	—	.0078	—	
	do	do	do	—	—	Fall 1956	—	1/.352	—	—	—	—	
	do	do	do	—	—	Fall 1957	1.0	—	—	—	—	—	
	do	do	do	—	—	Fall 1958	1.0	—	—	—	.0591	—	
	do	do	do	—	—	Fall 1959	1.0	—	—	—	0	—	
	do	do	do	—	—	Fall 1960	1.0	—	—	—	.0158	—	
	do	do	do	—	—	Fall 1961	1.0	—	—	—	0	—	
	do	do	do	—	—	Fall 1962	1.0	—	—	—	.0065	—	
	do	do	do	—	—	Fall 1963	1.0	—	—	—	.3228	—	
	do	do	do	—	—	Fall 1964	1.0	—	—	—	.0889	—	
	do	do	do	—	—	Fall 1965	1.0	—	—	—	.0054	—	
71-32a	Teakettle Reservoir No. 2a	do	do	.273	.273	Fall 1938	—	.079	—	—	—	—	FS
	do	do	do	—	—	Fall 1941	3.0	.068	—	—	.0146	—	
	do	do	do	—	—	Fall 1948	7.0	.050	—	—	.0084	—	
	do	do	do	—	—	Fall 1951	3.0	.030	—	—	.0280	—	
	do	do	do	—	—	Fall 1955	4.0	.015	—	—	.0110	—	
	do	do	do	—	—	Fall 1956	1.0	2/.066	—	—	—	—	
	do	do	do	—	—	Fall 1957	1.0	—	—	—	—	—	
	do	do	do	—	—	Fall 1958	1.0	—	—	—	.0241	—	
	do	do	do	—	—	Fall 1959	1.0	—	—	—	.0059	—	
	do	do	do	—	—	Fall 1960	1.0	—	—	—	0	—	
	do	do	do	—	—	Fall 1961	1.0	—	—	—	0	—	
	do	do	do	—	—	Fall 1962	1.0	—	—	—	.0003	—	
	do	do	do	—	—	Fall 1963	1.0	—	—	—	.0462	—	
	do	do	do	—	—	Fall 1964	1.0	—	—	—	.0372	—	
	do	do	do	—	—	Fall 1965	1.0	—	—	—	0	—	
71-33a	Teakettle Reservoir No. 2	do	do	.647	.647	Fall 1938	—	.134	—	—	—	—	FS
	do	do	do	—	—	Fall 1948	10.0	.078	—	—	.0066	—	
	do	do	do	—	—	Fall 1951	3.0	.021	—	—	.0250	—	
	do	do	do	—	—	Fall 1955	4.0	.0022	—	—	.0080	—	
	do	do	do	—	—	Fall 1956	—	2/.101	—	—	—	—	
	do	do	do	—	—	Fall 1957	1.0	—	—	—	—	—	
	do	do	do	—	—	Fall 1958	1.0	—	—	—	.0166	—	
	do	do	do	—	—	Fall 1959	1.0	—	—	—	0	—	
	do	do	do	—	—	Fall 1960	1.0	—	—	—	.0032	—	
	do	do	do	—	—	Fall 1961	1.0	—	—	—	.0032	—	
	do	do	do	—	—	Fall 1962	1.0	—	—	—	.0015	—	
	do	do	do	—	—	Fall 1963	1.0	—	—	—	.0474	—	
	do	do	do	—	—	Fall 1964	1.0	—	—	—	0	—	
	do	do	do	—	—	Fall 1965	1.0	—	—	—	—	—	
71-34a	Teakettle Reservoir No. 3	do	do	.856	.856	Fall 1938	—	.131	—	—	—	—	FS
	do	do	do	—	—	Fall 1940	2.0	.127	—	—	.0036	—	
	do	do	do	—	—	Fall 1941	1.0	.109	—	—	.0026	—	
	do	do	do	—	—	Fall 1948	7.0	.040	—	—	.0112	—	
	do	do	do	—	—	Fall 1951 3/	3.0	.007	—	—	.0160	—	
	do	do	do	—	—	Fall 1956	—	2/.117	—	—	—	—	
	do	do	do	—	—	Fall 1957	1.0	—	—	—	—	—	
	do	do	do	—	—	Fall 1958	1.0	—	—	—	.0073	—	
	do	do	do	—	—	Fall 1959	1.0	—	—	—	.0062	—	
	do	do	do	—	—	Fall 1960	1.0	—	—	—	.0003	—	
	do	do	do	—	—	Fall 1961	1.0	—	—	—	0	—	
	do	do	do	—	—	Fall 1962	1.0	—	—	—	.0049	—	
	do	do	do	—	—	Fall 1963	1.0	—	—	—	.1457	—	
	do	do	do	—	—	Fall 1964	1.0	—	—	—	.0619	—	
	do	do	do	—	—	Fall 1965	1.0	—	—	—	.0338	—	
71-35	Teakettle No. 7	do	do	.091	.091	Fall 1956	(1938-56)	.222	.139	—	—	—	FS
	do	do	do	—	—	Fall 1957	19.0	.181	.129	—	.029	—	
	do	do	do	—	—	Fall 1958	1.0	.178	.127	—	4/-.013	—	
	do	do	do	—	—	Fall 1959	1.0	.182	.130	—	4/-.037	—	
	do	do	do	—	—	Fall 1960	1.0	.178	.127	—	.069	—	

71-36	Pine Flat	Kings River	Sanger, Calif.	1,542	1,542	May	1952	--	1,013,400	.612	--	--	--	CE
	do	do	do	--	--	Feb.	1954	5/1.8	1,013,290	.612	*62	.04	54	
	do	do	do	--	--	Nov.	1956	6/3	1,011,950	.611	*62	.29	392	
71-37	Salinas Boys Ranch	Trib. of Natividad Creek	Salinas, Calif.	.13	.13	Oct.	19537/	--	7/9.65	--	--	--	--	SCS
	do	do	do	--	--	Sept.	1964	11	9.34	--	*65	.23	325.6	
71-38	Roy Alexander	do	do	.205	.203	--	19518/	--	8/8.40	--	--	--	--	SCS
	do	do	do	--	--	Sept.	1964	13	8.15	--	*65	.10	141.6	
71-39	Gibralter	Santa Ynez	Santa Barbara, Calif.	216	6/202.2	Oct.	1919	--	15,296	.42	--	--	--	GS
	do	do	do	--	--	Aug.	1923	3.83	13,674	.38	--	1.96	--	
	do	do	do	--	--	Aug.	1944	21	7,720	.21	--	9/1.47	--	
	do	do	do	--	--	Feb.	1956	11.42	14,777	.41	--	.43	--	
	do	do	do	--	--	Aug.	1969	13.5	9,654	.27	--	1.88	--	
71-40	Santa Felicia	Piru Creek	Piru, Calif.	425	425	Oct.	1955	--	101,200	--	--	--	--	GS
	do	do	do	--	--	Oct.	1965	10	98,730	--	52	.58	658	
71-41	Success Lake	Tule River	Porterville, Calif.	393	393	Sept.	1960	--	*86,160	.681	--	--	--	CE
	do	do	do	--	--	Nov.	1965	5.2	86,160	.681	--	*0	*0	
	do	do	do	--	--	Nov.	1967	2.0	83,610	.660	*62	3.244	4,383	
	do	do	do	--	--	Dec.	1968	1.1	83,680	.661	*62	-.163	-218.7	
71-42	Isabella Reservoir	Kern River	Isabella, Calif.	2,074	2,074	June	1953	--	539,665	.81	--	--	--	CE
	do	do	do	--	--	Sept.	1956	3.3	539,115	.81	*62	.074	100	
	do	do	do	--	--	Dec.	1968	12.2	534,465	.803	*62	.186	252	
71-43	Lake Kaweah (Terminus Dam)	Kaweah River	Lemon Cove, Calif.	560	560	Nov.	1961	--	149,599	.389	--	--	--	CE
	do	do	do	--	--	Nov.	1967	6	147,099	.382	*62	.745	1,006	
71-44	Cachuma	Santa Ynes River	Santa Ynez, Calif.	417	196	Jan.	1953	--	204,874	--	--	--	--	BR
	do	do	do	--	--	Aug.	1969	16.58	10/192,574	--	--	10/3.79	--	

SACRAMENTO, EEL AND RUSSIAN RIVER BASINS

72-1a	Ridgewood (Walker)	Walker Creek	Willits, Calif.	5.7	5.6	--	1930	--	313	.10	--	--	--	SCS
	do	do	do	--	--	June	1966	36	216	.07	77.5	.48	1,063	
72-2a	Morris	Davis Creek	do	5.1	5.03	--	1924	--	*320	.12	--	--	--	SCS
	do	do	do	--	--	--	1927	3	*775	.29	--	--	--	
	do	do	do	--	--	May	1960	36	660	.24	*50	.63	686	
	do	do	do	--	--	May	1966	6	643	.24	50.3	.56	642	
72-3	Big Canyon	Big Canyon Creek	French Town, Calif.	5.50	5.48	Nov.	1934	--	200	--	--	--	--	SCS
	do	do	do	--	--	Oct.	1945	11	196	--	44.7	.069	67.2	
72-4	Blodgett	Trib. of Cosumnes River	Sacramento, Calif.	3.12	3.05	Mar.	1940	--	258	--	--	--	--	SCS
	do	do	do	--	--	Oct.	1945	5.6	254	--	46	.217	217	
72-5	Bullards Bar	North Yuba River	Camptonville, Calif.	480	479	Oct.	1919	--	31,500	--	--	--	--	SCS
	do	do	do	--	--	Jan.	1939	19.2	28,893	--	*70	.284	433	
72-6	Combie (Van Geisen)	Bear River	Auburn, Calif.	130	129	June	1928	--	8,545	--	--	--	--	SCS
	do	do	do	--	--	Oct.	1935	7.3	7,840	--	*70	.749	1,140	
72-7b	East Park	Little Stony Creek	Stonyford, Calif.	11/101.5	98.9	Dec.	1910	--	12/50,900	--	--	--	--	GS
	do	do	do	--	--	Nov.	1962	52	48,940	.60	55	13/.37	13/443	
72-8	Faulke Lake (False Lake)	N. Fk. Jenney Creek	Shasta, Calif.	.71	.68	--	1851	--	130	--	--	--	--	SCS
	do	do	do	--	--	Dec.	1945	94	120	--	54	.147	173	
72-9	Gerber	Trib. of Burch Creek	Corning, Calif.	.31	.28	June	1917	--	190	--	--	--	--	SCS
	do	do	do	--	--	Dec.	1945	28.5	182	--	78.3	.971	1,656	
72-10	Magalia	Little Butte Creek	Chico, Calif.	14/8.23	8.08	Jan.	1918	--	3,718	--	--	--	--	SCS
	do	do	do	--	--	Jan.	1946	28	3,648	--	48.8	.307	326	
72-11b	Stony Gorge	Stony Creek	Elk Creek, Calif.	15/199	197	Nov.	1928	--	16/50,000	--	--	--	--	GS
	do	do	do	--	--	Nov.	1962	34.1	48,160	.29	54	13/.27	13/319	
72-12	Misselbeck	N. Fk. Cottonwood Creek	Redding, Calif.	12.0	11.8	May	1920	--	4,300	--	--	--	--	SCS
	do	do	do	--	--	Dec.	1945	25.5	4,086	--	*75	.711	1,161	
72-13	Lake Pillsbury (Scott Dam)	Eel River	Potter Valley, Calif.	283	284	Dec.	1921	--	94,396	.248	--	--	--	GS
	do	do	do	--	--	May	1959	37.5	86,785	.228	73	.71	1,129	
72-14	Catacoula (Bar 49)	Maxwell Creek	St. Helena, Calif.	.71	.69	--	1953	--	183.0	--	--	--	--	SCS
	do	do	do	--	--	Nov.	1958	5	181.4	--	--	.46	--	
72-15	Milliken	Milliken Creek	Napa, Calif.	10.5	10.4	--	1924	--	2,000	--	--	--	--	SCS
	do	do	do	--	--	Oct.	1958	34	1,988	--	--	.04	--	
72-16a	Onion Creek No. 1	Onion Creek, trib. of Amer. River.	Soda Springs, Calif.	.19	.19	--	1957	--	.114	.039	--	--	--	FS
	do	do	do	--	--	--	1958	1.0	.096	.033	--	.074	--	
	do	do	do	--	--	--	1959	1.0	.097	.033	--	.071	--	
	do	do	do	--	--	--	1960	1.0	.093	.032	--	.024	--	
72-17	Onion Creek No. 2	do	do	.48	.48	--	1957	--	.132	.042	--	--	--	FS
	do	do	do	--	--	--	1958	1.0	.083	.026	--	.1226	--	
	do	do	do	--	--	--	1959	1.0	.081	.026	--	.0004	--	
	do	do	do	--	--	--	1960	1.0	.069	.022	--	.0206	--	
72-18	Onion Creek No. 3	do	do	.65	.65	--	1958	--	.191	.062	--	--	--	FS
	do	do	do	--	--	--	1959	1.0	.205	.067	--	17/.0000	--	
	do	do	do	--	--	--	1960	1.0	.186	.061	--	.0273	--	

1/ Overtopped by sediment, no measurements taken in 1956; sediment removed down to original reservoir bottom.

2/ Sediment removed in summer 1956 - no measurements taken.

3/ Last data for which significant data are available; reservoir inoperative until 1956 when sediment was removed.

4/ Loss of sediment not applicable to watershed as a whole.

5/ Period of abnormal operation. Represents 0.3 year of sediment accumulation behind partially closed gates and 1.5 years of sediment depletion with wide open gates. Sediment value is residue at end of period.

6/ Represents sedimentation for 3 full runoff years because no significant inflow occurred from Oct. 1, 1953, to Feb. 13, 1954, or from Oct. 1, 1956, to Nov. 15, 1956.

7/ Original capacity determined from 1965 survey.

8/ Original capacity determined from 1964 survey.

9/ Effective drainage area prior to 1931 was 216 sq. mi. There after 202.2 sq. mi.

10/ Data at elevation 748 (limit of 8-7-69 survey). Spillway crest elevation 750 ft.

11/ Natural drainage area. Flow diverted into the reservoir from Stony Gorge drainage basin after 1914.

12/ Spillway elevation (flashboard crest elevation 50,900); USGS Water-Supply Paper 1798-F.

13/ Based on the natural drainage area.

14/ Excluding 3 sq. mi. above P. G. & E. Canal.

15/ The natural drainage area less area above East Park Reservoir. Flow diverted from the basin to East Park Reservoir after 1914.

16/ U. S. Bureau of Reclamation capacity curve (3-31-45); USGS Water-Supply Papers 1635 Part II., 1798-F.

17/ Net sediment loss - treated as 0 gain.

* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY		PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET							AC.-FT.	TONS	
			SACRAMENTO, EEL AND RUSSIAN RIVER BASINS (Continued)											
72-19a	Onion Creek No. 5	Onion Creek, trib. of Amer. River.	Soda Springs, Calif.	0.39	0.39	—	1957	—	0.177	0.038	—	—	—	FS
	do	do	do	—	—	—	1958	1.0	.112	.037	—	0.1260	—	
	do	do	do	—	—	—	1959	1.0	.101	.033	—	.0256	—	
	do	do	do	—	—	—	1960	1.0	.046	.015	—	.1750	—	
72-20a	Onion Creek No. 7	do	do	.80	.80	—	1958	—	.0734	.025	—	—	—	FS
	do	do	do	—	—	—	1959	1.0	.0723	.024	—	.0069	—	
	do	do	do	—	—	—	1960	1.0	.0605	.020	—	.0080	—	
72-21	Leo Trombadore	Trib. of Dry Creek	Geyserville, Calif.	.038	.037	Aug.	1961 1/	—	1/10.30	*.221	—	—	—	SCS
	do	do	do	—	—	Oct.	1965	4	9.86	*.211	*66	2.97	4,269	
72-22	Hazel Hill	do	do	.175	.173	Oct.	1960 1/	—	1/21.50	*.099	—	—	—	SCS
	do	do	do	—	—	Oct.	1965	5	21.04	*.097	*65	.52	736	
72-23	Babe Wood	do	Cloverdale, Calif.	1.69	1.68	Oct.	1958 1/	—	1/54.85	*.027	—	—	—	SCS
	do	do	do	—	—	Nov.	1965	7	46.76	*.023	*78.9	.69	1,186	
72-24	Nielsen	Trib. of Santa Rosa Creek	Santa Rosa, Calif.	.187	.184	Nov.	1960 3/	—	3/17.36	.092	—	—	—	SCS
	do	do	do	—	—	Nov.	1964	4	16.62	.088	*65	.96	1,359	
72-25	Leo Frediani	Seven Oaks Creek	Healdsburg, Calif.	.594	.592	Oct.	1960 3/	—	3/6.78	*.010	—	—	—	SCS
	do	do	do	—	—	Oct.	1965	5	6.61	*.007	4/67	.73	1,385	
72-26a	Casper Creek - South Fork	Casper Creek	Casper, Calif.	1.63	1.63	Fall	1962	—	—	—	—	—	—	FS
	do	do	do	—	—	Summer	1963	.80	—	—	—	.054	—	
	do	do	do	—	—	Summer	1964	1.0	—	—	—	.028	—	
	do	do	do	—	—	Summer	1965	1.0	—	—	—	.139	—	
	do	do	do	—	—	Summer	1966	1.0	—	—	—	.119	—	
	do	do	do	—	—	Summer	1967	1.0	—	—	—	.066	—	
	do	do	do	—	—	Summer	1968	1.0	—	—	—	.083	—	
	do	do	do	—	—	Summer	1969	1.0	—	—	—	.143	—	
	do	do	do	—	—	Summer	1970	1.0	—	—	—	.091	—	
72-27a	Casper Creek - North Fork	do	Fort Bragg, Calif.	1.92	1.92	Fall	1962	—	—	—	—	—	—	FS
	do	do	do	—	—	Summer	1963	.78	—	—	—	.079	—	
	do	do	do	—	—	Summer	1964	1.0	—	—	—	.063	—	
	do	do	do	—	—	Summer	1965	1.0	—	—	—	.425	—	
	do	do	do	—	—	Summer	1966	1.0	—	—	—	.491	—	
	do	do	do	—	—	Summer	1967	1.0	—	—	—	.037	—	
	do	do	do	—	—	Summer	1968	1.0	—	—	—	.033	—	
	do	do	do	—	—	Summer	1969	1.0	—	—	—	.262	—	
	do	do	do	—	—	Summer	1970	1.0	—	—	—	.141	—	
72-28	McGuire	Virgin Creek	Fort Bragg, Calif.	.084	.083	July	1954 5/	—	7.3	.11	—	—	—	SCS
	do	do	do	—	—	July	1967	13	7.07	.11	*60	.21	274	
72-29	Lazy Creek	Lazy Creek	Philo, Calif.	.666	.663	Aug.	1955 5/	—	26.96	.038	—	—	—	SCS
	do	do	do	—	—	July	1967	12	19.50	.027	67	.96	1,372	
72-30	Appleton	Unnamed	Healdsburg, Calif.	.166	.161	Aug.	1950 5/	—	25.18	.189	—	—	—	SCS
	do	do	do	—	—	Aug.	1967	17	26.26	.182	59	.37	475	
72-31	M. S. Wilson	Bourns Gulch	Gualala, Calif.	.285	.283	July	1952 5/	—	14.06	*.040	—	—	—	SCS
	do	do	do	—	—	July	1967	15	13.02	*.037	*60	.24	324	
			KLAMATH, ROGUE, AND UMPQUA RIVER BASINS											
73-1	Emigrant Gap	Emigrant Creek	Ashland, Oreg.	61.6	61.2	Oct.	1924	—	6/8,300	—	—	—	—	BR
	do	do	do	—	—	Oct.	1951	27	7,500	—	—	.314	—	
73-2	Coyote Creek #1	Trib. South Umpqua River	Tiller, Oreg.	.267	.267	June	1965	—	7/.019	.0001	—	—	—	FS
	do	do	do	—	—	Aug.	1966	1.12	—	—	—	.0052	—	
	do	do	do	—	—	Aug.	1967	1.01	—	—	—	.0035	—	
	do	do	do	—	—	Aug.	1968	1.0	—	—	—	.0016	—	
	do	do	do	—	—	July	1969	.94	—	—	—	.0028	—	
	do	do	do	—	—	July	1970	.98	—	—	—	.0019	—	
73-3	Coyote Creek #2	do	do	.264	.264	June	1965	—	7/.012	.00004	—	—	—	FS
	do	do	do	—	—	Aug.	1966	1.11	—	—	—	.0011	—	
	do	do	do	—	—	Aug.	1967	1.01	—	—	—	.0003	—	
	do	do	do	—	—	Aug.	1968	1.0	—	—	—	.0004	—	
	do	do	do	—	—	July	1969	.94	—	—	—	.0002	—	
	do	do	do	—	—	July	1970	.98	—	—	—	.0004	—	
73-4	Coyote Creek #3	do	do	.192	.192	June	1965	—	7/.011	.0001	—	—	—	FS
	do	do	do	—	—	Aug.	1966	1.11	—	—	—	.0047	—	
	do	do	do	—	—	Aug.	1967	1.01	—	—	—	.002	—	
	do	do	do	—	—	Aug.	1968	1.0	—	—	—	.0023	—	
	do	do	do	—	—	July	1969	.94	—	—	—	.001	—	
	do	do	do	—	—	July	1970	.98	—	—	—	.0057	—	

73-5	Coyote Creek #4	Trib. South Umpqua River	Tiller, Oreg.	.188	.188	June	1965	—	7/—	—	—	—	—	FS
	do	do	do	—	—	Aug.	1966	1.11	—	—	—	.0094	—	
	do	do	do	—	—	Aug.	1967	1.01	—	—	—	.0009	—	
	do	do	do	—	—	Aug.	1968	1.0	—	—	—	.0027	—	
	do	do	do	—	—	July	1969	.94	—	—	—	.001	—	
	do	do	do	—	—	July	1970	.98	—	—	—	.0049	—	

LOWER COLUMBIA RIVER BASIN AND PACIFIC COAST BASINS IN NORTHERN OREGON

74-1	Condit (White Salmon)	White Salmon River	Underwood, Wash.	337	337	—	1913	—	8/1,081	—	—	—	—	SCS
	do	do	do	—	—	May	1936	23	1,054	—	—	.004	—	
74-2	Lake Harriet (Oak Grove)	Trib. of Clackamas River	Portland, Oreg.	126	126	Aug.	1924	—	256	.001	—	—	—	SCS
	do	do	do	—	—	Aug.	1948	24	252	.001	—	.0013	—	
74-3	McKay	McKay Creek	Pendleton, Oreg.	186	184	Oct.	1926	—	73,737	—	—	—	—	SCS
	do	do	do	—	—	July	1946	19.75	73,474	—	*60	.07	61	
74-4	Cottage Grove	Coast Fk. Willamette River	Cottage Grove, Oreg.	104	102	Oct.	1942	—	33,000	*.165	—	—	—	CE
	do	do	do	—	—	Dec.	1947	5.1	32,873	*.164	—	.244	—	
74-5a	Luther Claypool Pond	Trib. of Beaver Creek	Post, Oreg.	1.09	1.09	—	1946	—	3.79	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1951	5	3.59	—	*70	.037	56.4	
74-6	Paul Jaeger No. 1 Pond	Trib. of John Day River	Condon, Oreg.	.625	.624	—	1943	—	1.82	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1951	8	1.60	—	*70	.085	130	
74-7	Paul Jaeger No. 2 Pond	Trib. of Rock Creek	do	.468	.467	—	1946	—	3.19	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1951	5	2.91	—	*70	.299	456	
74-8a	Paul Jaeger No. 3 Pond	Trib. of John Day River	do	.250	.249	—	1944	—	3.04	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1951	7	2.82	—	*70	.12	183	
74-9	J. M. Wilson Pond	Trib. of Deschutes River	Moro, Oreg.	1.90	1.90	—	1946	—	1.08	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1951	5	.97	—	*70	.01	15.24	
74-10	Willard Barnet Pond	Buck Creek	Kent, Oreg.	.210	.209	—	1941	—	2.91	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1951	10	2.61	—	*70	.24	366	
74-11	Arthur Schmidt Pond	Trib. of Buck Creek	Shaniko, Oreg.	.115	.114	—	1940	—	2.00	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1951	11	1.97	—	*70	.03	45.73	
74-12	Rock Creek Improvement Co.	Trib. of White River	Tygh Valley, Oreg.	5.62	5.60	—	1938	—	290	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1951 9/	13	267	—	*70	.065	68.61	
74-13	Vernon Christ Pond	Five Mile Creek	Dufur, Oreg.	.089	.088	—	1941	—	1.61	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1951	10	1.39	—	*70	.227	346	
74-14	Milt J. Martin Pond	Mill Creek	The Dalles, Oreg.	.070	.069	—	1944	—	2.72	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1951	7	2.58	—	*70	.29	442	
74-15	Hunt Livestock Co. Pond	Trib. of Buck Creek	Shaniko, Oreg.	.090	.050	—	1942	—	2.25	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1951	9	2.15	—	*70	.20	305	
74-16	Cold Springs	Cold Springs Canyon	Cold Springs, Oreg.	188	186	Feb.	1908	—	49,709	—	—	—	—	SCS
	do	do	do	—	—	May	1951	43	44,668	—	78	.63	1,070	
74-17	Dorena	Row River	Cottage Grove, Oreg.	265	262	Oct.	1949	—	77,900	.143	—	—	—	CE
	do	do	do	—	—	Sept.	1958	8.9	77,150	.142	—	.150	—	
74-18	H. J. Andrews #1	Trib. Blue River	Blue River, Oreg.	.37	.37	Aug.	1965	—	7/.12	.0001	—	—	—	FS
	do	do	do	—	—	Aug.	1966	1.05	—	—	—	.021	—	
	do	do	do	—	—	July	1967	.92	—	—	—	.214	—	
	do	do	do	—	—	Aug.	1968	.99	—	—	—	.243	—	
	do	do	do	—	—	Aug.	1969	.84	—	—	—	.332	—	
	do	do	do	—	—	Aug.	1970	.96	—	—	—	.095	—	
74-19	H. J. Andrews #2	do	do	.233	.233	Aug.	1965	—	7/.056	.0001	—	—	—	FS
	do	do	do	—	—	Aug.	1966	.98	—	—	—	.0311	—	
	do	do	do	—	—	July	1967	.92	—	—	—	0	—	
	do	do	do	—	—	Aug.	1968	.99	—	—	—	.0013	—	
	do	do	do	—	—	Aug.	1969	1.0	—	—	—	.0257	—	
	do	do	do	—	—	Aug.	1970	1.0	—	—	—	.0255	—	
74-20	H. J. Andrews #3	do	do	.391	.391	Dec.	1965	—	7/.061	.00005	—	—	—	FS
	do	do	do	—	—	Apr.	1966	.31	—	—	—	—	—	
	do	do	do	—	—	Aug.	1967	.97	—	—	—	.054	—	
	do	do	do	—	—	Aug.	1968	1.0	—	—	—	.041	—	
	do	do	do	—	—	Aug.	1969	1.0	—	—	—	.10	—	
	do	do	do	—	—	Aug.	1970	.96	—	—	—	.076	—	

1/ Original capacity determined by 1965 survey.
2/ Original capacity determined from 1964 survey. Capacity shown is for an elevation 4 ft. below spillway crest.
3/ Original capacity obtained during 1965 survey. Total capacity with flashboards = 9.41 ac.-ft.
4/ From one disturbed and two undisturbed samples.
5/ Original capacity determined from 1967 survey.
6/ Based on original topographic survey; believed to be unreliable.
7/ Sediment measured and removed periodically.
8/ Dam was raised 5 feet in 1927.
9/ Built in 1941. Reservoir was thoroughly cleaned in 1945.
* Estimated or assumed.

SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER ACRE-FT)	SPECIFIC WEIGHT (DRY) (LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
	COLUMBIA RIVER BASIN (GRAND COULEE TO UMATILLA) AND PACIFIC COAST DRAINAGE IN WASHINGTON Yakima, Chelan, and Okanogan River Basins												
75-1	Diablo	Skagit River	Bellingham, Wash.	1,100	1,100	— 1930	—	89,220	—	—	—	—	SCS
	do	do	do	—	—	July 1936	6	88,862	—	—	0.054	—	
75-2	High Valley Ranch #1 Pond	Wenas Creek	Yakima, Wash.	4.11	4.10	— 1939	—	8.61	—	—	—	—	SCS
	do	do	do	—	—	Oct. 1951	12	7.63	—	*70	.02	30.49	
75-3	High Valley Ranch #2 Pond	do	Ellensburg, Wash.	.184	.184	— 1940	—	.98	—	—	—	—	SCS
	do	do	do	—	—	Oct. 1951	11	.51	—	*70	.04	61.0	
75-4	High Valley Ranch #3 Pond	do	do	.312	.312	— 1940	—	1.53	—	—	—	—	SCS
	do	do	do	—	—	Oct. 1951	11	1.42	—	*70	.03	46.0	
75-5	Henry Clerf Pond	Rye Grass Creek	do	6.47	6.47	— 1946	—	2.38	—	—	—	—	SCS
	do	do	do	—	—	Oct. 1951	5	2.02	—	*70	.01	15.24	
75-6	Coffin Sheep Co. Pond	Naches River	Yakima, Wash.	.482	.480	— 1939	—	2.63	—	—	—	—	SCS
	do	do	do	—	—	Oct. 1951	12	2.47	—	*70	.027	41.16	
75-7	Fox Creek	Fox	Ardenvoir, Wash.	1.82	1.82	Summer 1965	—	1/—	—	—	—	—	FS
	do	do	do	—	—	Summer 1966	1	—	—	110	.0076	18.4	
	do	do	do	—	—	Summer 1967	1	—	—	—	.0076	18.4	
	do	do	do	—	—	Summer 1968	1	—	—	—	.0022	5.3	
	do	do	do	—	—	Summer 1969	1	—	—	—	.0076	18.4	
	do	do	do	—	—	Summer 1970	1	—	—	—	0	0	
	do	do	do	—	—	Summer 1971	1	—	—	—	.0577	136.5	
75-8	Burns	Burns	do	2.18	2.18	Summer 1965	—	1/—	—	—	—	—	FS
	do	do	do	—	—	Summer 1966	1	—	—	—	.0	0	
	do	do	do	—	—	Summer 1967	1	—	—	—	.0	0	
	do	do	do	—	—	Summer 1968	1	—	—	—	.0	0	
	do	do	do	—	—	Summer 1969	1	—	—	—	.0	0	
	do	do	do	—	—	Summer 1970	1	—	—	—	.0	0	
	do	do	do	—	—	Summer 1971	1	—	—	110	.0366	87.9	
75-9	McCree	McCree	do	1.985	1.985	Summer 1965	—	1/—	—	—	—	—	FS
	do	do	do	—	—	Summer 1966	1	—	—	—	0	0	
	do	do	do	—	—	Summer 1967	1	—	—	—	0	0	
	do	do	do	—	—	Summer 1968	1	—	—	—	0	0	
	do	do	do	—	—	Summer 1969	1	—	—	—	0	0	
	do	do	do	—	—	Summer 1970	1	—	—	—	0	0	
	do	do	do	—	—	Summer 1971	1	—	—	110	.0165	39.7	
75-10	Crabtree	Trib. White	Auburn, Wash.	.475	.11	May 1967	—	6.92	.01	—	—	—	SCS
	do	do	do	—	—	June 1970	3	6.38	.009	*60	1.65	2,156	
75-11	Llewellyn	Trib. Goose	Wilbur, Wash.	1.25	1.25	July 1957	—	17.52	.26	—	—	—	SCS
	do	do	do	—	—	July 1967	10	8.62	.13	*80	.73	1,272	
75-12	Schwisow	No name	Ritzville, Wash.	4.15	4.15	May 1959	—	3.54	.04	—	—	—	SCS
	do	do	do	—	—	May 1969	10	0	—	*80	.09	156.8	
75-13	Rex Lyle	do	do	.71	.71	June 1959	—	2.75	.2	—	—	—	SCS
	do	do	do	—	—	May 1969	10	1.66	.12	*80	.155	270	
75-14	Schoessler	do	do	.31	.31	June 1951	—	2.24	.37	—	—	—	SCS
	do	do	do	—	—	May 1969	18	0	—	*80	.40	696.8	
75-15	John Landa (PU 90, BLK. 81)	do	Moses Lake, Wash.	.092	.092	Jan. 1970	—	2/—	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1970	1	—	—	*60	.8	1,045	
75-16	Shaffer	do	do	.356	.356	June 1958	—	2/—	—	—	—	—	SCS
	do	do	do	—	—	Nov. 1970	12	—	—	*60	.28	366	
75-17	Sun Spiced No. 1	do	do	.17	.17	— 1969	—	2/—	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1970	1	—	—	*60	.73	954	
75-18	Sun Spiced No. 2	do	do	.80	.80	June 1969	—	2/—	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1970	1	—	—	*60	.52	680	
75-19	Sheffels	do	Wilbur, Wash.	4.77	4.77	Apr. 1963	—	5.1	.02	—	—	—	SCS
	do	do	do	—	—	May 1969	6	.85	.003	*70	.15	229	
75-20	Telecky	Trib. Crab	Edwall, Wash.	2.26	2.26	Apr. 1957	—	13.43	.09	—	—	—	SCS
	do	do	do	—	—	June 1964	7	0	—	*80	.85	1,480	
75-21	Terwilliger (PU 273, BLK. 1)	No Name	Warden, Wash.	.05	.05	Apr. 1968	—	2/—	—	—	—	—	SCS
	do	do	do	—	—	Dec. 1968	1	—	—	*60	1.4	1,829	
	do	do	do	—	—	Dec. 1969	1	—	—	*60	8.6	11,238	
75-22	Roby	Goose Creek	Davenport, Wash.	4.10	3.85	— 1938	—	22.91	.10	—	—	—	SCS
	do	do	do	—	—	Apr. 1969	30	2.56	.01	*70	.175	266	

COLUMBIA RIVER BASIN (INTERNATIONAL BOUNDARY TO GRAND COULEE) AND PACIFIC COAST DRAINAGE IN WASHINGTON
Pendoreille, Spokane, Walla Walla, and Lower Snake River Basins

76-1	Mrs. Weise No. 1 Pond	Austin Creek	Austin, Wash.	.129	.128	—	1965	—	1.45	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1951	6	1.23	—	*70	.31	473	
76-2	Mrs. Weise No. 2 Pond	Trib. of Austin Creek	do	.218	.217	—	1965	—	2.54	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1951	6	2.14	—	*70	.31	473	
76-3a	Lester Reeves No. 1 Pond	Austin Creek N. & S. Fork	do	.101	.100	—	1961	—	1.45	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1951	10	1.18	—	*70	.27	4,116	
76-4	Lester Reeves No. 2 Pond	Austin Creek	do	.125	.125	—	1945 3/	—	1.21	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1951	6	.69	—	*70	.70	1,067	
76-5	Sam Holt Pond	Latah Creek	Waverly, Wash.	.110	.109	—	1944	—	2.30	—	—	—	—	SCS
	do	do	do	—	—	Oct.	1951	7	2.11	—	*70	.12	183	
76-6	Ray T. Wood Pond	Trib. of Clearwater River	Lewiston, Idaho	.014	.013	—	1946	—	4.22	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1951	5	3.87	—	*70	5.38	8,202	
76-7	Ed Knowlton Pond	Clearwater River	Orofino, Idaho	.021	.020	—	1946	—	2.54	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1951	5	2.12	—	*70	4.20	6,403	
76-8	Weldon Wasson Pond	do	Nez Perce, Idaho	.015	.014	—	1945	—	4.58	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1951	6	4.40	—	*70	2.14	3,263	
76-9	Henry Kortemeier Pond	Palouse River	Potlatch, Idaho	.021	.020	—	1943	—	2.71	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1951	8	2.52	—	*70	1.20	1,830	
76-10	George Heidel Pond	do	Harvard, Idaho	.043	.042	—	1941	—	2.49	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1951	10	2.41	—	*70	.19	290	
76-11a	Johanna Nelson Pond	do	Deary, Idaho	.014	.012	—	1943	—	5.16	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1951	8	4.97	—	*70	2.0	3,050	
76-12	Ed Galloway Pond	do	do	.014	.014	—	1944	—	1.29	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1951	7	1.23	—	*70	.64	976	
76-13	A. E. Tweedy Pond	Clearwater River	Nez Perce, Idaho	.014	.014	—	1931	—	.80	—	—	—	—	SCS
	do	do	do	—	—	Sept.	1951	20	.62	—	*70	.64	976	
76-14	Walla Walla River Arm McNary Reservoir	Walla Walla River	Wallula, Wash.	1,760	1,760	Apr.	1953	—	9,688	.023	—	—	—	CE
	do	do	do	—	—	Nov.	1956	3.61	7,237	.017	*75	.405	662	
	do	do	do	—	—	Nov.	1958	2.00	5,907	.013	—	.484	791	
	do	do	do	—	—	Nov.	1960	2.00	4,656	.011	—	.255	417	
	do	do	do	—	—	Sept.	1961	.83	3,569	.008	—	.744	1,215	
	do	do	do	—	—	Sept.	1962	1.00	3,498	.008	—	.073	119	
	do	do	do	—	—	Sept.	1963	1.00	2,918	.007	*75	.307	501	
	do	do	do	—	—	Oct.	1964	1.08	2,874	.007	—	.023	38	
	do	do	do	—	—	July	1965	.75	1,912	.006	—	4/1.027	4/1,678	
76-15	Main Fork Horse Creek	Horse Creek	Elk City, Idaho	6.51	6.51	Oct.	1965	—	1/.087	—	—	—	—	FS
	do	do	do	—	—	Oct.	1966	1	—	—	*90	.0024	4.8	
	do	do	do	—	—	Oct.	1967	1	—	—	*90	.0064	12.9	
	do	do	do	—	—	Oct.	1968	1	—	—	*90	.0024	4.7	
	do	do	do	—	—	Oct.	1969	1	—	—	*90	.0043	8.5	
	do	do	do	—	—	Oct.	1970	1	—	—	*90	.0048	9.4	
	do	do	do	—	—	Oct.	1971	1	—	—	*90	.0134	26.2	
76-16	East Fork Horse Creek	East Fork Horse Creek	do	5.56	5.56	Oct.	1965	—	1/.039	—	—	—	—	FS
	do	do	do	—	—	Oct.	1966	1	—	—	*90	.0022	4.1	
	do	do	do	—	—	Oct.	1967	1	—	—	*90	.0056	11.0	
	do	do	do	—	—	Oct.	1968	1	—	—	*90	.0016	3.8	
	do	do	do	—	—	Oct.	1969	1	—	—	*90	.0029	5.6	
	do	do	do	—	—	Oct.	1970	1	—	—	*90	.0045	8.8	
	do	do	do	—	—	Oct.	1971	1	—	—	*90	.0070	13.7	

COLUMBIA RIVER BASIN IN CANADA

77-

SNAKE RIVER BASIN (FROM KING HILL TO GRANDE RONDE RIVER)

78-1	Orchard	Indian Creek	Boise, Idaho	43	42.8	—	1892	—	4,737	—	—	—	—	SCS
	do	do	do	—	—	June	1947	55	4,668	—	—	.03	—	
78-2	Black Canyon	Payette River	Emmett, Idaho	2,750	5/2,580	June	1924	—	37,699	.017	—	—	—	SCS
	do	do	do	—	—	June	1936	12	33,622	.015	—	.132	—	
78-3	Pleasant Valley	Black's Creek	Boise, Idaho	*16	*16	—	1905	—	7,097	—	—	—	—	SCS
	do	do	do	—	—	June	1947	42	7,055	—	—	.063	—	
78-4	Arrowrock-Boise Project	Boise River	do	2,211	2,170	Feb.	1915	—	279,250	.186	—	—	—	BR
	do	do	do	—	—	Oct.	1947	32.6	271,550	.181	73	.109	173.3	
78-5	Andy Anderson Pond	Weiser River	Cambridge, Idaho	.614	.590	—	1905	—	104.20	—	—	—	—	SCS
	do	do	do	—	—	Aug.	1952	46	100.67	—	*70	.136	207	
78-6	Hilton Branch Pond	Trib. of Weiser River	do	.436	.424	—	1940	—	44.49	—	—	—	—	
	do	do	do	—	—	Sept.	1951	11	43.24	—	*70	.259	395	

1/ Sediment measured and removed periodically.
2/ Desilting structure for return irrigation water.
3/ Built 1941. Reservoir was thoroughly cleaned in 1945.

4/ Includes above crest deposits within original flow lines.
5/ Does not include drainage area above Payette Lakes.
* Estimated or assumed.

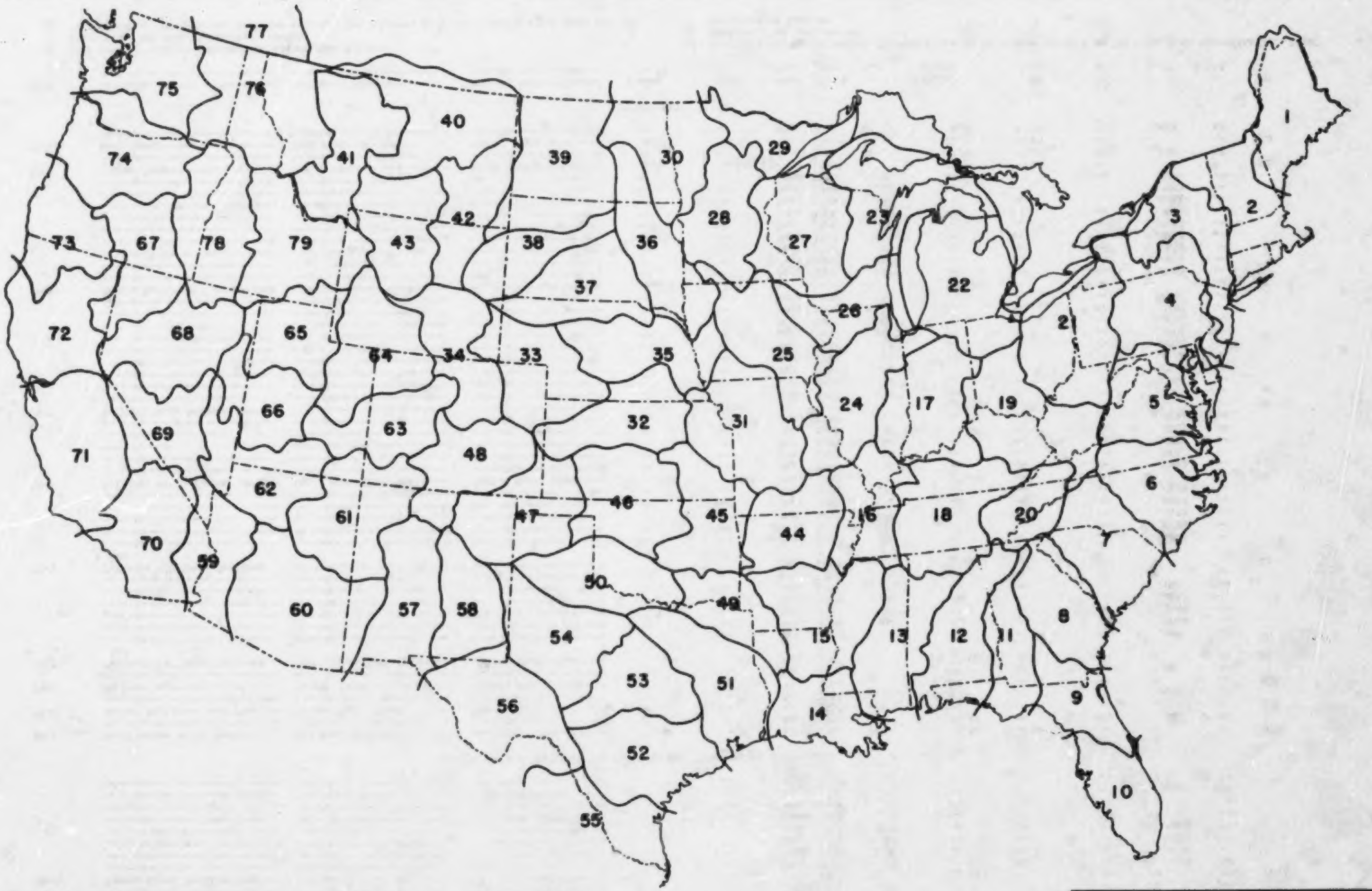
SUMMARY OF
RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1970

DATA SHEET NUMBER	RESERVOIR	STREAM	NEAREST TOWN	DRAINAGE AREA (SQUARE MILES)		DATE OF SURVEY	PERIOD BETWEEN SURVEYS (YEARS)	STORAGE CAPACITY (ACRE-FT.)	CAPACITY AVG. ANN. INFLOW RATIO (ACRE-FT. PER, ACRE-FT)	SPECIFIC WEIGHT (DRY) LB. PER CU. FT.)	AVG. ANN. SEDIMENT ACCUMULATION PER SQ. MI. OF NET DR. AREA FOR PERIOD SHOWN		AGENCY SUPPLYING DATA
				TOTAL	NET						AC.-FT.	TONS	
				SNAKE RIVER BASIN (FROM KING HILL TO GRANDE RONDE RIVER) (Continued)									
78-7	W. B. Wimminger Pond	Trib. of Weiser River	Cambridge, Idaho	0.343	0.338	— 1943	—	17.58	—	—	—	—	SCS
	do	do	do	—	—	Aug. 1951	8	16.57	—	*70	0.240	366	
78-8	Miller Sneed Pond	Little Willow Creek	Payette, Idaho	.218	.217	— 1945	—	2.90	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1951	6	2.71	—	*70	.147	234	
78-9	Twin Puddles Pond	Trib. of Boise River	Boise, Idaho	1.61	1.61	— 1941	—	7.82	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1951	10	7.52	—	*70	.02	30.49	
78-10	Lambkin Pond	do	Mountain Home, Idaho	.193	.191	— 1941	—	4.54	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1951	10	4.49	—	*70	.05	76.23	
78-11	Mud Springs Pond	do	do	1.07	1.06	— 1939	—	12.18	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1951	12	11.81	—	*70	.05	43.73	
78-12	J. J. Colton Pond	Trib. of Powder River	Keating, Oreg.	.484	.483	— 1939	—	1.08	—	—	—	—	SCS
	do	do	do	—	—	Oct. 1951	12	.84	—	*70	.04	41.0	
78-13	Hankins Pond No. 1	Trib. of Clover Creek	Westfall, Oreg.	.12	—	— 1952	—	1.56	—	—	—	—	SCS
	do	do	do	—	—	May 1956	4	1.52	—	—	.079	—	
78-14	Hankins Pond No. 2	do	do	.27	—	— 1952	—	6.94	—	—	—	—	SCS
	do	do	do	—	—	May 1956	4	6.76	—	—	.17	—	
78-15a	Wheaton	Wheaton Creek	do	14.0	—	— 1948	—	52.27	—	—	—	—	SCS
	do	do	do	—	—	May 1956	8	51.97	—	—	.18	—	
78-16	E-1 Creek Sediment Dam	Trib. Silver Creek	Cascade, Idaho	.098	.098	Aug. 1965	—	1/.008	—	—	—	—	FS
	do	do	do	—	—	June 1966	.80	—	—	—	.0002	—	
	do	do	do	—	—	June 1967	.98	—	—	—	.0007	—	
	do	do	do	—	—	July 1968	1.09	—	—	—	.0047	—	
	do	do	do	—	—	Nov. 1968	.35	—	—	—	.0003	—	
	do	do	do	—	—	June 1969	.60	—	—	—	.0409	—	
	do	do	do	—	—	Aug. 1969	.21	—	—	—	0	—	
	do	do	do	—	—	July 1970	.84	—	—	—	.0194	—	
	do	do	do	—	—	Sept. 1970	.21	—	—	—	0	—	
78-17	Central Creek Sediment Dam	do	do	.775	.775	Aug. 1965	—	1/.013	—	—	—	—	FS
	do	do	do	—	—	June 1966	.79	—	—	—	.0002	—	
	do	do	do	—	—	June 1967	1.0	—	—	—	.0014	—	
	do	do	do	—	—	July 1968	1.07	—	—	—	.0006	—	
	do	do	do	—	—	Nov. 1968	.36	—	—	—	.0011	—	
	do	do	do	—	—	June 1969	.62	—	—	—	.0079	—	
	do	do	do	—	—	Aug. 1969	.17	—	—	—	0	—	
	do	do	do	—	—	June 1970	.83	—	—	—	.0038	—	
	do	do	do	—	—	Sept. 1970	.21	—	—	—	0	—	
78-18	Cabin Creek Sediment Dam	do	do	.40	.40	July 1965	—	1/.046	—	—	—	—	FS
	do	do	do	—	—	June 1966	.87	—	—	—	.0032	—	
	do	do	do	—	—	June 1967	.99	—	—	—	.0009	—	
	do	do	do	—	—	July 1968	1.07	—	—	—	.0038	—	
	do	do	do	—	—	Nov. 1968	.37	—	—	—	.0041	—	
	do	do	do	—	—	June 1969	.60	—	—	—	.0416	—	
	do	do	do	—	—	Aug. 1969	.19	—	—	—	0	—	
	do	do	do	—	—	June 1970	.82	—	—	—	.0162	—	
	do	do	do	—	—	Sept. 1970	.21	—	—	—	0	—	
78-19	Ditch Creek Sediment Dam	do	do	.41	.41	Sept. 1965	—	1/.023	—	—	—	—	FS
	do	do	do	—	—	June 1966	.75	—	—	—	.0036	—	
	do	do	do	—	—	June 1967	1.0	—	—	—	.0107	—	
	do	do	do	—	—	July 1968	1.07	—	—	—	.0039	—	
	do	do	do	—	—	Nov. 1968	.36	—	—	—	.002	—	
	do	do	do	—	—	June 1969	.56	—	—	—	.067	—	
	do	do	do	—	—	Aug. 1969	.23	—	—	—	0	—	
	do	do	do	—	—	June 1970	.82	—	—	—	.0214	—	
	do	do	do	—	—	Sept. 1970	.21	—	—	—	0	—	
78-20	"C" Creek Sediment Dam	do	do	.75	.75	Aug. 1965	—	1/.0207	—	—	—	—	FS
	do	do	do	—	—	June 1966	.80	—	—	—	.0103	—	
	do	do	do	—	—	June 1967	.98	—	—	—	.037	—	
	do	do	do	—	—	July 1968	1.08	—	—	—	.00211	—	
	do	do	do	—	—	Oct. 1968	.30	—	—	—	.0177	—	
	do	do	do	—	—	June 1969	.60	—	—	—	.04	—	
	do	do	do	—	—	Aug. 1969	.24	—	—	—	0	—	
	do	do	do	—	—	June 1970	.82	—	—	—	.0276	—	
	do	do	do	—	—	Sept. 1970	.21	—	—	—	0	—	
78-21	"D" Creek Sediment Dam	do	do	.47	.47	July 1965	—	1/.017	—	—	—	—	FS
	do	do	do	—	—	June 1966	.85	—	—	—	.0017	—	
	do	do	do	—	—	June 1967	.99	—	—	—	.0187	—	

	do	do	do	—	—	July 1968	1.09	—	—	—	.002	—	
	do	do	do	—	—	Nov. 1968	.35	—	—	—	.0036	—	
	do	do	do	—	—	June 1969	.57	—	—	—	.0194	—	
	do	do	do	—	—	Aug. 1969	.22	—	—	—	0	—	
	do	do	do	—	—	June 1970	.82	—	—	—	.0025	—	
	do	do	do	—	—	Sept. 1970	.21	—	—	—	0	—	
78-22	Eggers Creek Sediment Dam	do	do	.5	.5	Aug. 1965	—	1/.013	—	—	—	—	FS
	do	do	do	—	—	June 1966	.81	—	—	—	.0012	—	
	do	do	do	—	—	June 1967	1.01	—	—	—	.0054	—	
	do	do	do	—	—	July 1968	1.08	—	—	—	.0013	—	
	do	do	do	—	—	Nov. 1968	.35	—	—	—	.0012	—	
	do	do	do	—	—	June 1969	.58	—	—	—	2/0	—	
	do	do	do	—	—	Aug. 1969	.21	—	—	—	0	—	
	do	do	do	—	—	June 1970	.82	—	—	—	.013	—	
	do	do	do	—	—	Sept. 1970	.21	—	—	—	0	—	
			SNAKE RIVER BASIN (ABOVE KINGS HILL) AND SALMON RIVER BASIN										
79-1	Rattlesnake Butte Pond	Big Wood River	Gooding, Idaho	4.81	4.80	— 1943	—	25.56	—	—	—	—	SCS
	do	do	do	—	—	Sept. 1951	8	23.89	—	*70	.04	61	
79-2	Salmon Falls	Salmon Falls Creek	Rogerson, Idaho	1,560	1,555	— 1910	—	229,650	—	—	—	—	SCS
	do	do	do	—	—	Oct. 1955	45.0	227,282	—	*50	.034	37	
79-3	Pioneer	Clover Creek	Glenns Ferry, Idaho	183	183	— 1904	—	1,264	—	—	—	—	SCS
	do	do	do	—	—	Aug. 1954	50±	898	—	—	.04	—	
79-4	Tailholt "A" Creek Sediment Dam	Trib. South Fork Salmon	Yellow Pine, Idaho	.845	.84	Aug. 1967	—	1/.0164	—	—	—	—	FS
	do	do	do	—	—	May 1968	.74	—	—	—	.001	—	
	do	do	do	—	—	Oct. 1968	.34	—	—	—	.0002	—	
	do	do	do	—	—	May 1969	.63	—	—	—	.0199	—	
	do	do	do	—	—	Sept. 1969	.34	—	—	—	.0017	—	
	do	do	do	—	—	June 1970	.72	—	—	—	.017	—	
	do	do	do	—	—	Sept. 1970	.25	—	—	—	0	—	
79-5	Tailholt "B" Creek Sediment Dam	do	do	.61	.61	Aug. 1967	—	1/.012	—	—	—	—	FS
	do	do	do	—	—	May 1968	.74	—	—	—	.0015	—	
	do	do	do	—	—	Oct. 1968	.40	—	—	—	.0086	—	
	do	do	do	—	—	May 1969	.64	—	—	—	.0156	—	
	do	do	do	—	—	Sept. 1969	.34	—	—	—	.0067	—	
	do	do	do	—	—	June 1970	.72	—	—	—	.0199	—	
79-6	Tailholt "C" Sediment Dam	Trib. South Fork Salmon	Yellow Pine, Idaho	.556	.548	Aug. 1967	—	1/.015	—	—	—	—	FS
	do	do	do	—	—	May 1968	.74	—	—	—	.0039	—	
	do	do	do	—	—	Oct. 1968	.54	—	—	—	.001	—	
	do	do	do	—	—	May 1969	.62	—	—	—	.0273	—	
	do	do	do	—	—	Sept. 1969	.27	—	—	—	.0009	—	
	do	do	do	—	—	June 1970	.72	—	—	—	.0134	—	
79-7	Tailholt Main Sediment Dam	do	do	2.54	3/2.50	— —	—	1/.0552	—	—	—	—	FS
	do	do	do	—	—	June 1966	.44	—	—	—	.0046	—	
	do	do	do	—	—	Sept. 1966	.29	—	—	—	.0019	—	
	do	do	do	—	—	May 1967	.68	—	—	—	.012	—	
	do	do	do	—	—	Sept. 1967	.33	—	—	—	.0148	—	
	do	do	do	—	—	May 1968	.66	—	—	—	.0082	—	
	do	do	do	—	—	Oct. 1968	.40	—	—	—	.0002	—	
	do	do	do	—	—	May 1969	.64	—	—	—	.0191	—	
	do	do	do	—	—	Oct. 1969	.35	—	—	—	.0037	—	
	do	do	do	—	—	June 1970	.60	—	—	—	.0187	—	
79-8	Circle End Creek Sediment Dam	do	do	1.452	1.452	— —	—	1/.013	—	—	—	—	FS
	do	do	do	—	—	Apr. 1966	.58	—	—	—	.0169	—	
	do	do	do	—	—	Sept. 1966	.42	—	—	—	.0016	—	
	do	do	do	—	—	May 1967	.65	—	—	—	.0199	—	
	do	do	do	—	—	Sept. 1967	.35	—	—	—	.0163	—	
	do	do	do	—	—	May 1968	.65	—	—	—	.0052	—	
	do	do	do	—	—	Oct. 1968	.40	—	—	—	.0046	—	
	do	do	do	—	—	Apr. 1969	.54	—	—	—	.0144	—	
	do	do	do	—	—	Sept. 1969	.46	—	—	—	.0024	—	
	do	do	do	—	—	June 1970	.70	—	—	—	.0036	—	
79-9	Indian Fork	Indian Fork	Ririe, Idaho	1.6	1.6	— 1939*	—	3.5	*.007	—	—	—	SCS
	do	do	do	—	—	Sept. 1966	41	1.88	*.004	95.3	.025	52	
79-10	Rock Creek	Rock Creek	Ammon, Idaho	6.76	6.76	— 1946*	—	12.6	*.008	—	—	—	SCS
	do	do	do	—	—	Aug. 1966	20	8.96	*.006	79.8	.027	45	
			PUERTO RICO										
80-1	Caonillas	Caonillas River	Utuado, Puerto Rico	4/48.9	47.9	June 1948	—	5/50,149	.966	—	—	—	SCS
	do	do	do	—	—	Nov. 1952	4.3	49,883	.960	38.1	1.39	1,070	

1/ Pond used to take bedload samples and organic matter.
2/ All values lost due to incorrect survey.
3/ Total drainage for this dam includes drainage for Tailholt A, B and C Creeks after 8-22-67.

4/ Caonillas extension project increased the drainage area to 85.5 sq. mi.; put into operation Oct. 1952.
5/ Determined by 1952 survey.
* Estimated or assumed.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79

Index number corresponds to first of two numbers in summary table, which appear in column headed "Data Sheet No."

INDEX OF RIVER BASIN MAPS

RESERVOIR SEDIMENT DATA SUMMARY

CONCHAS RESERVOIR
NAME OF RESERVOIR

47-1b
DATA SHEET NO.

DAM	1. OWNER Corps of Engineers	2. STREAM Canadian and Conchas	3. STATE New Mexico
	4. SEC. 33 TWP. 14N RANGE 26E	5. NEAREST P O. Conchas Dam	6. COUNTY San Miguel
	7. LAT 35° 24' 10" LONG 104° 11' 25"	8. TOP OF DAM ELEVATION 4240	9. SPILLWAY CREST ELEV. 4201 1/

RESERVOIR	10. STORAGE ALLOCATION	11. ELEVATION TOP OF POOL	12. ORIGINAL SURFACE AREA, ACRES	13. ORIGINAL CAPACITY, ACRE-FEET	14. GROSS STORAGE, ACRE-FEET	15. DATE STORAGE BEGAN
	a. FLOOD CONTROL	4,218	13,715	201,834	601,112	1 Jan. 1939
	b. MULTIPLE USE					
	c. POWER					
	d. WATER SUPPLY					16. DATE NORMAL OPER. BEGAN
	e. IRRIGATION					
	f. CONSERVATION	4,201	10,073	296,412	399,278	Jan. 1939
	g. INACTIVE	4,155	2,520	102,866	102,866	

17. LENGTH OF RESERVOIR Canadian 23 2/ MILES | AV. WIDTH OF RESERVOIR 4200 Contour 0.75 MILES

WATERSHED		
	18. TOTAL DRAINAGE AREA 7,409 SQ. MI.	22. MEAN ANNUAL PRECIPITATION 15.2 3/ INCHES
	19. NET SEDIMENT CONTRIBUTING AREA 6,976 SQ. MI.	23. MEAN ANNUAL RUNOFF 0.4864 4/ (65.6) INCHES
	20. LENGTH 100 MILES AV. WIDTH 73 MILES	24. MEAN ANNUAL RUNOFF 192,200 (65.6) AC.-FT.
	21. MAX. ELEV. 13,000 MIN. ELEV. 4,074	25. ANNUAL TEMP MEAN 49° RANGE 20-75 3/

SURVEY DATA

26. DATE OF SURVEY 5/	27. PERIOD YEARS	28. ACCL. YEARS	29. TYPE OF SURVEY	30. NO. OF RANGES OR CONTOUR INT.	31. SURFACE AREA, ACRES	32. CAPACITY, ACRE-FEET	33. C/I. RATIO, AC.-FT. PER AC.-FT.
Jan. 1939			Contour	10 feet	13,715	601,112	3.13
May 1940	1.4	1.4	Range	14 ranges		599,712	3.12
June 1942	2.1	3.4	Range	24 ranges		585,112	3.04
Nov. 1942	.4	3.8	Range	28 ranges		581,112	3.02
Oct. 1944	1.9	5.7	Contour	10 feet	13,349	576,756	3.00
Feb. 1949	4.3	10.1	Contour	10 feet	13,552	566,16	2.95
Oct. 1963	14.7	24.83	Range (D)	45 ranges	13,677	550,79	2.87
Oct. 1970	6.92	31.75	Contour	5 feet	13,664	528,951	2.75

26. DATE OF SURVEY 5/	34. PERIOD ANNUAL PRECIPITATION	35. PERIOD WATER INFLOW, ACRE-FEET			36. WATER INFL. TO DATE, AC.-FT.	
		a. MEAN ANNUAL	b. MAX. ANNUAL	c. PERIOD TOTAL	a. MEAN ANNUAL	b. TOTAL TO DATE
	Inches					
May 1940	14.26	72,700		101,780	72,700	101,780
June 1942	22.40	859,780	1,059,699	1,805,540	560,980	1,907,320
Nov. 1942	12.50	1,079,980		431,990	615,607	2,339,310
Oct. 1944	14.32	143,750	1,168,350	273,130	458,320	2,612,440
Feb. 1949	13.68	120,500	158,858	518,140	309,960	3,130,580
Oct. 1963	13.01	144,340	336,514	2,121,850	211,790	5,252,430
Oct. 1970	12.04	157,100	394,190	1,087,100	199,670	6,339,530

26. DATE OF SURVEY 5/	37. PERIOD CAPACITY LOSS, ACRE-FEET			38. TOTAL SED. DEPOSITS TO DATE, ACRE-FEET		
	a. PERIOD TOTAL	b. AV. ANNUAL	c. PER SQ. MI.-YEAR	a. TOTAL TO DATE	b. AV. ANNUAL	c. PER SQ. MI.-YEAR
May 1940	1,400			1,400	1,000	0.143
June 1942	14,600	6,952	0.996	16,000	4,710	.675
Nov. 1942	4,000			20,000	5,260	.754
Oct. 1944	4,356	2,290	.328	24,356	4,270	.612
Feb. 1949	10,593	2,460	.353	34,949	3,460	.496
Oct. 1963	15,364	1,045	.150	50,313	2,030	.291
Oct. 1970	21,848	3,157	.453	72,161	2,273	.326

26. DATE OF SURVEY 5/	39. AV. DRY WGT., LBS. PER CU. FT.	40. SED. DEP., TONS PER SQ. MI.-YR.		41. STORAGE LOSS, PCT.		42. SED. INFLOW, PPM	
		a. PERIOD	b. TOTAL TO DATE	a. AV. ANN.	b. TOT. TO DATE	a. PERIOD	b. TOT. TO DATE
May 1940	75.7*		236	0.17	0.23	16,687	16,687
June 1942	75.7	1,643	1,113	.78	2.66	9,810	10,177
Nov. 1942	75.7		1,243	.88	3.33	11,233	10,372
Oct. 1944	75.7	541	1,009	.71	4.05	19,348	11,310
Feb. 1949	75.7	581	818	.58	5.81	24,802	12,543
Oct. 1963	75.7	247	479	.34	8.37	8,748	11,621
Oct. 1970	75.7	746	537	.38	12.00	24,376	12,808

*Estimated

26. DATE OF SURVEY	43. DEPTH DESIGNATION RANGE IN FEET BELOW, AND ABOVE, CREST ELEVATION											
	178-128	128-108	108-88	88-68	68-58	58-48	48-38	38-28	28-Crest	Crest-17	17-29	
	PERCENT OF TOTAL SEDIMENT LOCATED WITHIN DEPTH DESIGNATION											
May 1940												
June 1942												
Nov. 1942												
Oct. 1944	16	4	7	10	6	10	15	16	16			
Feb. 1949	13	4	5	9	5	7	11	14	14	16	2	
Oct. 1963	9	9	8	15	9	11	13	11	8	7		
Oct. 1970	3	7	12	10	5	6	8	11	33	4	1	

26. DATE OF SURVEY	44. REACH DESIGNATION PERCENT OF TOTAL ORIGINAL LENGTH OF RESERVOIR														
	0-10	10-20	20-30	30-40	40-50	50-60	60-70	70-80	80-90	90-100	-105	-110	-115	-120	-125
	PERCENT OF TOTAL SEDIMENT LOCATED WITHIN REACH DESIGNATION														
May 1940 6/	114	22	3	-26	-16	2				S. Canadian Arm					
	30	22	7	10	18	13				Conchas Arm					
June 1942	6	3	4	4	22	14	27	18	2	S. Canadian Arm					
	20	21	-12	12	13	27	5	14		Conchas Arm					
Nov. 1942	6	2	5	8	16	16	29	16	2	S. Canadian Arm					
	-1	0	1	2	19	57	5	14	3	Conchas Arm					

45. RANGE IN RESERVOIR OPERATION

WATER YEAR	MAX. ELEV.	MIN. ELEV.	INFLOW, AC.-FT.	WATER YEAR	MAX. ELEV.	MIN. ELEV.	INFLOW, AC.-FT.
1947	4,202.46	4,199.00	129,330	1959	4,200.38	4,193.40	112,620
1948	4,201.46	4,195.63	154,700	1960	4,199.37	4,192.68	131,520
1949	4,200.97	4,192.50	158,260	1961	4,201.75	4,196.55	216,440
1950	4,198.59	4,185.85	131,410	1962	4,201.13	4,193.03	119,380
1951	4,194.68	4,184.24	106,770	1963	4,193.01	4,176.16	76,510
1952	4,184.15	4,168.23	125,930	1964	4,178.58	4,156.05	31,060
1953	4,176.16	4,162.07	107,950	1965	4,201.83	4,157.61	394,190
1954	4,173.22	4,155.80	32,030	1966	4,200.77	4,192.35	108,660
1955	4,190.37	4,157.10	297,760	1967	4,195.35	4,185.79	142,740
1956	4,189.98	4,173.19	51,880	1968	4,193.13	4,182.51	112,130
1957	4,175.40	4,163.50	129,930	1969	4,193.65	4,180.65	192,830
1958	4,201.82	4,173.97	336,510	1970	4,197.55	4,189.30	105,630

46. ELEVATION-AREA-CAPACITY DATA

ELEVATION	AREA	CAPACITY	ELEVATION	AREA	CAPACITY	ELEVATION	AREA	CAPACITY
4,230	16,380	709,119	4,180	5,513	173,912	4,110	311	1,299
4,220	14,110	556,724	4,170	4,323	125,103	4,100	2	24
4,218	13,664	528,951	4,160	3,394	86,519	4,090	1	9
4,210	11,845	426,866	4,150	2,642	56,348	4,080		2
4,201	9,692	330,124	4,140	1,959	33,495	4,070	0	0
4,200	9,463	320,546	4,130	1,323	17,170	4,060	0	0
4,190	7,290	237,119	4,120	797	6,690			

47. REMARKS AND REFERENCES

1/ Emergency Spillway Crest at 4218.
2/ Conchas 12.8 miles.
3/ From climatic Atlas dated June 1968.
4/ This figure affected by water taken out above reservoir for irrigation.
5/ Totals computed to end of each month shown.
6/ Only dates computed.

48. AGENCY MAKING SURVEY Albuquerque District Corps of Engineers
49. AGENCY SUPPLYING DATA Corps of Engineers
50. DATE May 1970

April 1966

RESERVOIR SEDIMENT
DATA SUMMARY

NAME OF RESERVOIR

DATA SHEET NO.

DAM	1. OWNER	2. STREAM	3. STATE
	4. SEC. TWP. RANGE	5. NEAREST P.O.	6. COUNTY
	7. LAT. ° ' " LONG. ° ' "	8. TOP OF DAM ELEVATION	9. SPILLWAY CREST ELEV.

RESERVOIR	10. STORAGE ALLOCATION	11. ELEVATION TOP OF POOL	12. ORIGINAL SURFACE AREA, ACRES	13. ORIGINAL CAPACITY, ACRE-FEET	14. GROSS STORAGE, ACRE-FEET	15. DATE STORAGE BEGAN
	a. FLOOD CONTROL					
	b. MULTIPLE USE					
	c. POWER					
	d. WATER SUPPLY					16. DATE NORMAL OPER. BEGAN
	e. IRRIGATION					
	f. CONSERVATION					
	g. INACTIVE					
	17. LENGTH OF RESERVOIR MILES			AV. WIDTH OF RESERVOIR MILES		

WATERSHED	18. TOTAL DRAINAGE AREA SQ. MI.		22. MEAN ANNUAL PRECIPITATION INCHES
	19. NET SEDIMENT CONTRIBUTING AREA SQ. MI.		23. MEAN ANNUAL RUNOFF INCHES
	20. LENGTH MILES	AV. WIDTH MILES	24. MEAN ANNUAL RUNOFF AC.-FT.
	21. MAX. ELEV.	MIN. ELEV.	25. ANNUAL TEMP: MEAN RANGE

SURVEY DATA

26. DATE OF SURVEY	27. PERIOD YEARS	28. ACCL. YEARS	29. TYPE OF SURVEY	30. NO. OF RANGES OR CONTOUR INT.	31. SURFACE AREA, ACRES	32. CAPACITY, ACRE-FEET	33. C/I. RATIO, AC.-FT. PER AC.-FT.

26. DATE OF SURVEY	34. PERIOD ANNUAL PRECIPITATION	35. PERIOD WATER INFLOW, ACRE-FEET			36. WATER INFL. TO DATE, AC.-FT.	
		a. MEAN ANNUAL	b. MAX. ANNUAL	c. PERIOD TOTAL	a. MEAN ANNUAL	b. TOTAL TO DATE

26. DATE OF SURVEY	37. PERIOD CAPACITY LOSS, ACRE-FEET			38. TOTAL SED. DEPOSITS TO DATE, ACRE-FEET		
	a. PERIOD TOTAL	b. AV. ANNUAL	c. PER SQ. MI.-YEAR	a. TOTAL TO DATE	b. AV. ANNUAL	c. PER SQ. MI.-YEAR

26. DATE OF SURVEY	39. AV. DRY WGT., LBS. PER CU. FT.	40. SED. DEP., TONS PER SQ. MI.-YR.		41. STORAGE LOSS, PCT.		42. SED. INFLOW, PPM	
		a. PERIOD	b. TOTAL TO DATE	a. AV. ANN.	b. TOT. TO DATE	a. PERIOD	b. TOT. TO DATE

26. DATE OF SURVEY	43. DEPTH DESIGNATION RANGE IN FEET BELOW, AND ABOVE, CREST ELEVATION											
	PERCENT OF TOTAL SEDIMENT LOCATED WITHIN DEPTH DESIGNATION											

26. DATE OF SURVEY	44. REACH DESIGNATION PERCENT OF TOTAL ORIGINAL LENGTH OF RESERVOIR														
	0-10	10-20	20-30	30-40	40-50	50-60	60-70	70-80	80-90	90-100	-105	-110	-115	-120	-125
	PERCENT OF TOTAL SEDIMENT LOCATED WITHIN REACH DESIGNATION														

45. RANGE IN RESERVOIR OPERATION

WATER YEAR	MAX. ELEV.	MIN. ELEV.	INFLOW, AC.-FT.	WATER YEAR	MAX. ELEV.	MIN. ELEV.	INFLOW, AC.-FT.

46. ELEVATION-AREA-CAPACITY DATA

ELEVATION	AREA	CAPACITY	ELEVATION	AREA	CAPACITY	ELEVATION	AREA	CAPACITY

47. REMARKS AND REFERENCES

48. AGENCY MAKING SURVEY

49. AGENCY SUPPLYING DATA

50. DATE ______________

USDA-SCS-HYATTSVILLE, MD. 1966

April 1966

REFERENCES FOR APPENDIX F

Miscellaneous Publication No. 1266, U.S. Department of Agriculture, Sedimentation Laboratory, Oxford, Mississippi, July, 1973.

APPENDIX G

EQUIVALENTS OF COMMONLY USED UNITS OF MEASUREMENT

TABLE G-1

EQUIVALENTS OF COMMONLY USED UNITS OF MEASUREMENT

English Unit	Multiplier	SI Unit	English Unit	Multiplier	SI Unit
acre	X4.046.724→ ← 2.471X10^{-4}X	m^{2}	gpd/ft	X0.0124→ ← 80.65X	m^{3}/day m
acre	X0.405→ ← 2.471X	ha*	gpd/sq ft	X0.0408→ ← 24.51X	m^{3}/day m^{2}
acre-ft	X1,233.5→ ← 8.11X10^{-4}X	m^{3}	gpm	X0.0631→ ← 15.85X	dm^{3}/s
Btu	X1.055→ ← 0.9478X	kJ	gpm	X0.0631→ ← 15.85X	l*/s
Btu	X0.252→ ← 3.968X	kg-cal*	gpm/sq ft	X40.7→ ← 0.0245X	l*/min m^{2}
Btu/hr/sq ft	X3.158→ ← 0.316X	J/s-m^{2}	hp	X0.7454→ ← 1.341X	kW
Btu/lb	X0.555→ ← 1.80X	kg-cal/kg*	hp-hr	X2.684→ ← 0.372X	MJ
cfm	X0.028→ ← 35.71X	m^{3}/min	in.	X2.54→ ← 0.3937X	cm
cfs	X1.7→ ← 0.588X	m^{3}/min	lb/day/acre-ft	X3.68→ ← 0.2717X	g/day m^{3}
cfs/sq miles	X0.657→ ← 1.522X	m^{3}/min km^{2}	lb/1,000 cu ft	X16.0→ ← 0.0625X	g/m^{3}
cu ft	X0.028→ ← 35.314X	m^{3}	lb/day/cu ft	X16→ ← 0.0625X	kg/day, m^{3}
cu ft	X28.32→ ← 0.0353X	l*	lb/mil gal	X0.92→ ← 8.333X	g/m^{3}
cu in.	X16.39→ ← 0.061X	cm^{3}	mil gal	X3,785→ ← 2.64X10^{-4}X	m^{3}
cu yd	X0.75→ ← 1.3709X	m^{3}	mgd	X3,785→ ← 2.64X10^{-4}X	m^{3}/day
°F	0.555(°F−32)→ ← 1.8(°C)+32	°C	mgd	X0.0438→ ← 22.82X	m^{3}/s
°C	plus 273→ ← minus 273	K	mile	X1.61→ ← 0.621X	km
ft	X0.3048→ ← 3.28X	m	ppb	X10^{-3}→ ← 1,000X	mg/l*
ft-lb	X1.356→ ← 0.737X	J	ppm	approximately equal to	mg/l*
gal	X3.785→ ← 0.264X	l*	sq ft	X0.0929→ ← 10.76X	m^{2}
gal	X0.003785→ ← 264.2X	m^{3}	sq in.	X645.2→ ← 0.00155	mm^{2}
gpd/acre	X0.9365→ ← 1.068X	m^{3}/day km^{2}	sq miles	X2.590→ ← 0.3861X	km^{2}

Other commonly used conversions:

1 MGD = 1.55cfs

γc_p = 62.4 BTU/ft^{3}/°F

1 MW = 3.414 X 10^{6} BTU/hr

1 BTU = 778 ft-lb

1 BTU = 252 cal

1 Langley/day = 3.7 BTU/ft.^{2}/day

*Not an SI unit, but a term commonly used and preferred as a wastewater unit of expression.